STUDIES IN STOCHASTIC MODELLING AND SIMULATION OF MIXED VEHICULAR TRAFFIC

STUDIES IN STOCHASTIC MODELLING AND SIMULATION OF MIXED VEHICULAR TRAFFIC

A Thesis Submitted
In Partial Fulfilment of the Requirements
for the Degree of
DOCTOR OF PHILOSOPHY

by
BALDEV RAJ MARWAH

to the

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY KANPUR
JULY, 1976

CERTIFICATE

This is to certify that the thesis "Studies in Stochastic Modelling and Simulation of Mixed Vehicular Traffic" submitted by Shri Baldev Raj Marwah in partial fulfilment of the requirements for the degree of Doctor of Philosophy of the Indian Institute of Technology, Kanpur, is a record of bonafide research work carried out by him under our supervision and guidance. The work embodied in this thesis has not been submitted elsewhere for a degree.

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

AR	Autoregressive process
ARMA	Autoregressive moving average process
ARIMA	Autoregressive integrated moving average process
A_j	Harmonic coefficient
AG(I)	Arrival gap of I-th vehicle
AT(I)	Arrival time of I-th vehicle
a	Arrival rate
a c f	Autocorrelation function
B	Backward shift operator
BC	Bullock cart
BY	Bicycle
B_j	Harmonic coefficient
CAT	Category of vehicle
C_j	Amplitude of cyclic component
D	Order of seasonal differencing; Density
D_j	Jam density
D_t	Density per km
D(I)	Available headway for I-th vehicle
d	Order of nonseasonal differencing
$\bar{d}$	Average delay per vehicle in seconds

e	Random error
e_t	Random component at time t
$f_1(t)$	Deterministic component at time t
$f_2(t)$	Persistence component at time t
h	Interarrival time gap in seconds
h_{min}	Minimum headway in seconds
IF, IF_i	Interaction factor for i-th category of vehicle in the mix
j	Cyclic index
KK	Tonga
KK(L)	Serial number of L-th vehicle in the lane
L	Lead time for forecast
LNGT	Length of section
N	Number of observations; Length of series
NCAT	Number of different categories of vehicles in the mix
NDEL(K)	Number of delayed vehicles of K-th category
NVT(I)	Stream logic of I-th vehicle
OS_{HT}	Operating speed of cars in homogeneous traffic
OS_{MTi}	Operating speed of cars in mixed traffic having i different categories of vehicles in the mix
OS_{Trucks}	Operating speed of trucks in homogeneous traffic
OT(I)	Overtaking time of I-th vehicle

P	Order of seasonal AR process
P_i	Volume of i-th category of vehicle expressed as percentage of car volume V_1
P_i'	Volume of i-th category of vehicle expressed as percentage of car truck volume $V_1 + V_2$
P(I)	Position of I-th vehicle
PCES(i)	Passenger car equivalent of i-th category of vehicle
PCUS(i)	Equivalent passenger car volume of mixed traffic having i different categories of vehicles
PFIN(I)	Position of I-th vehicle after overtaking
p	Order of nonseasonal AR process
p a c f	Partial autocorrelation function
Q	Order of seasonal MA process; Statistic
q	Order of nonseasonal MA process
r_k	Estimate of k-th order autocorrelation coefficient
SC	Scooter
S(I)	Minimum spacing of I-th vehicle
SD(I)	Maximum spacing for overtaking vehicle I
$S_w(L)$	Standard error of $w_t(L)$
s	Period of season
sd	Standard deviation
T	Truck
T_o	Headway gap
T_R	Reaction time of looking for gap

T_s	Time lag or stimulus response time in car-driver system
t_o	Travel time at no flow
$\overline{U}_f$	Mean free speed
$\overline{U}_s$	Average space mean speed
U_w	Wave velocity
u	Service rate
V	Traffic volume in VPH
VPH	Vehicles per hour in either direction
V_1	Car volume in VPH
V(I)	Running speed of I-th vehicle
VEF(I)	Operating speed of I-th vehicle
VF(I)	Free speed of I-th vehicle
w_t	Deviation of Z(t) from mean
$\hat{w}_t(L)$	L step ahead forecast of w_t series
$X_1(t)$	Log transformed trend free series
Z(t)	Traffic data series
$Z'(t)$	Moving average of Z(t) series
$Z_1(t)$	Trend free data series
α	Acceleration in kmph per sec.
α_j	Phase angle of cyclic component
∇	Difference operator
Θ, Θ_j	j-th order seasonal MA coefficient
θ, θ_j	j-th order MA coefficient

μ	Population mean
σ_a	Standard deviation of a_t series
Φ, Φ_j	j th order seasonal AR coefficient
ϕ, ϕ_j	j th order AR coefficient
χ^2	Chi square statistic
ψ_j	j th weight when AR process is expressed as weighted infinite sum of previous shocks

Abbreviations

AADT	Annual Average Daily Traffic
ADT	Average Daily Traffic
CRRI	Central Road Research Institute
DHV	Design Hourly Volume
IRC	Indian Roads Congress
MOT	Ministry of Transport
NCAER	National Council of Applied Economic Research
PCES	Passenger Car Equivalents
PCUS	Passenger Car Units

SYNOPSIS

"Studies in Stochastic Modelling and Simulation of Mixed Vehicular Traffic"- a thesis submitted in partial fulfilment of the requirements for the Degree of Doctor of Philosophy by Baldev Raj Marwah to the Department of Civil Engineering, Indian Institute of Technology, Kanpur, July, 1976.

Characterisation and estimation of traffic are two important aspects in the planning and design of a highway transportation system. The capacity of a highway is generally expressed in terms of passenger car volume per hour. In developing countries like India, the traffic is of mixed nature consisting of both slow and fast moving vehicles between which there are wide variations in speed. Ministry of Transport(MOT) and Central Road Research Institute(CRRI) in India specify the Passenger Car Equivalents (PCES) for different categories of vehicles. These PCES are considered constants and they do not account for the variations in traffic composition and volume.

This study considers the nonlinear interaction between the different vehicles in mixed traffic flow situations and attempts to characterise the variations of PCES in terms of traffic composition and volume. Traffic flow is stochastic in nature, the free speeds are probabilistic and the process is affected by logical decisions concerning

acceleration, retardation, overtaking etc. Analytic techniques of traffic flow theory like hydrodynamic analogy, car following theory, queuing theory etc., are suitable for the analysis of simple homogeneous vehicular traffic and it does not seem feasible to use exact analytic procedures for the analysis of complex mixed traffic flow. Computer simulation is adopted in this study to analyse the mixed traffic flow and infer the PCES under different volumes and compositions of traffic.

Traffic volume on a highway has hourly, daily and monthly variations and may have secular trends and random fluctuations. An understanding of the stochastic characteristics of traffic volume is needed to determine the design volume. Using historical monthly and daily traffic data of seven years on five main highways approaching Kanpur, stochastic models have been developed for the monthly and daily traffic data for three categories of goods carriers viz., incoming motor vehicles, transit motor vehicles and incoming slow moving vehicles. Two types of stochastic models were developed for the time series of monthly traffic data. They are:

(i) A general time series model of the form

$$Z_t = f_1(t) + f_2(t) + f_3(Z_{t-1}, Z_{t-2}, \ldots, Z_{t-k}) + e_t$$

where Z_t= traffic flow at time t; $f_1(t)$ = trend component; $f_2(t)$ = cyclic component, f_3= persistence component; and e_t = random component.

The secular trend in the monthly traffic series was indentified by moving average method and estimated by regression analysis. Trend was eliminated from the data series and the trend free series were subjected to correlogram and spectral analyses. It was found that 12 monthly cycle was significant in the truck (both incoming and transit) traffic and in addition 6 monthly cycle was also significant in the bullock cart traffic. The trend and cycle free data can be represented by a simple autoregressive model of first order. The parameters were estimated by standard procedures and this representation was found to be satisfactory for all the three categories of traffic on the five roads.

(ii) A general autoregressive moving average (ARMA) model can be represented by the equation $\phi_p(B)\ (Z_t - \bar{Z}) = \theta_q(B)\, e_t$, where Z_t= value of the variable at time t with a mean value $\bar{Z}$; e_t= random component; B= backward shift operator; and $\phi_p(B)$, $\theta_q(B)$ are polynominals in B of degrees p and q, representing respectively an autoregressive component of order p and moving average component of order q.

In case the time series is nonstationary, it is possible that the dth difference of $(Z_t-\bar{Z})$ may satisfy

the ARMA process. This is referred to as autoregressive integrated moving average (ARIMA) process. When seasonal effects are important, it may be possible to use a general multiplicative seasonal ARIMA model of order (p, d, q) x $(P, D, Q)_s$ given by

$$\Phi_P(B^s)\ \phi_p(B)\ \nabla_s^D\ \nabla^d\ (Z_t - \bar{Z}) = \Theta_Q(B^s)\,\theta_q(B)\ a_t$$

where in addition D= degree of seasonal differencing; s = period of season; ∇ = difference operator; $\Phi_P(B^s)$, $\Theta_Q(B^s)$ = polynomials in B^s of degrees P and Q respectively; and a_t is the white noise process. This model has the advantage that it requires fewer parameters than other models of comparable complexity.

For the monthly traffic series, multiplicative seasonal ARIMA model of order (1,0,0) x $(0,1,1)_{12}$ was found to fit the series well. The value of the various parameters of the model for three categories of vehicles on the five roads were determined by standard procedures. Tests for randomness were also conducted.

Daily fluctuations of traffic volume were also analysed for three representative months on two roads. Weekly differencing was found suitable for the daily series. The differenced series were found to follow a simple autoregressive process of first order. The stochastic models of

monthly and daily traffic can be used in estimation and forecasting of traffic demands.

Detailed data were needed for simulation and so field studies were conducted for ten days round the clock on a 2.25 km stretch of Grand Trunk Road. From the observations, peak hourly volumes; variations of volumes within the day of six vehicular types; and interarrival time gap distributions were estimated. The free speeds, delay time and operating speeds of different categories of vehicles were also estimated at varying volume levels and traffic compositions.

A computer simulation model of mixed traffic flow on a two lane highway was formulated. Vehicles move from either direction of the roadway section and scanning is done at one second intervals. Any vehicle in the roadway moved at its free speed where it had enough headway to travel unimpeded. In case it cannot move unimpeded and it has a higher speed than the vehicle ahead of it, then it may try to overtake. Overtaking is possible only if there is no conflict with the opposing traffic stream during overtaking operation. Otherwise the vehicle is forced to reduce its speed to that of the vehicle ahead until sometime later overtaking is possible. The above logic is applied to each of the vehicles in the section and those entering the section

for each time interval. As a vehicle leaves the section, time of leaving is noted and its characteristics like travel time, delay time and overall running speed etc., are determined.

Initially historical data of arrival times, traffic composition and system parameters were used to identify the system and validate the system model. The roadway is divided into one metre long sections and vehicles are moved from either direction at one second intervals. Vehicles move at their free speed when there is enough headway to travel unimpeded for two seconds. The change of speed of vehicles is assumed to be instantaneous and a looking for gap time of two seconds is provided for overtaking operations. The overtaking vehicle accelerates only if the speed difference with the overtaken vehicle is less than 16 kmph. Otherwise overtaking takes place at normal speed. For vehicles being overtaken, minimum spacing related to speed and length of vehicle, is specified. The results from the simulated model were consistent with the observed characteristics and the model is thus validated. Simulation is hence found to be a versatile tool for understanding of the flow process and identifying the system model.

Traffic on a highway is highly nonstationary with hourly trends, and cycles, and persistence. However, for design considerations a stationary peak rate corresponding

to a specified level of risk may be assumed. So this study deals with simulation of stationary traffic flow. It may be noted that when appropriate, more complicated nonstationary processes can be simulated to derive comparable results. Generated data were used in simulation of stationary mixed traffic flow at different volumes and compositions. When generated data are used in computer simulation, it is necessary to initialise the system. In this study initially the stretch of the road was assumed to be empty. The process was run until a steady state was reached. This initial time period varied generally with the volume level, being more for lower traffic volumes. Simulation was carried out further for a sufficiently long time for estimation of the characteristics of the process.

Simulation runs were performed for the following cases;

(i) Homogeneous traffic at different volumes for passenger cars and trucks taken separately.

(ii) Mixed traffic at varying volumes and traffic compositions for the following combinations:

(a) five cases consisting of two types of vehicles only, including passenger car and one of the other vehicles viz.,truck, motorbike, horse driven vehicle, bullock-cart, or bicycle;

(b) four cases consisting of combination of three vehicles including car, truck and one of the remaining four categories of vehicles; and

(c) all the six types of vehicles at the peak hourly compositions only.

On an average the processing time for one simulation run on IBM 7044/1401 system was more than half an hour. In all more than 100 hours of computer time was consumed.

When traffic is homogenous, the vehicles move essentially at their free speed at low volume levels. As the volume increases, the headways are reduced and there is a significant reduction in the operating speed of the vehicles till at a certain volume level, the density of the roadway section continues to increase thereby jamming the traffic. There is a nonlinear relationship between the volume level and the average operating speed of the vehicles. The capacity of a highway can thus be described for homogeneous traffic at any level of service(i.e., operating speed) by knowing the free speed distribution parameters of the vehicles.

In the case of mixed traffic flow, the effect on the operating speed of passenger cars was studied at varying traffic volumes and composition. The operating speed of the car in the mixed flow reduces even at low volume levels

due to restrained overtaking operations. The operating speed was also found to vary with the composition of slow moving vehicles in the mixed flow. A generalised conditional multiplicative model was developed to estimate the operating speed of cars in mixed traffic and the equivalent volumes of the homogeneous traffic at the same level of service were determined. Other vehicles affect the operating speed of cars and their PCES were estimated for combination of two or three types of vehicles at different volumes and compositions. The PCES so determined were not constants, but were found to vary nonlinearly with the traffic volume and composition. Curves indicating the relationships based on the generalised conditional multiplicative model are plotted and they may be used for design.

This study demonstrates that the daily and monthly traffic volume can be represented by stochastic models which can be used for estimation and forecasting of traffic demand. It also demonstrates that the complex problem of mixed traffic flow on a two lane highway can be mathematically modelled and analysed using computer simulation techniques. The results of this study may be used for design in the neighbour-hood of Kanpur and other comparable regions. For regions with significantly different traffic characteristics, the methodology demonstrated in this study may be used with appropriate modifications. The demand and capacity models can also be used in combination for the design of highway systems.

1. INTRODUCTION

1.1 General

Knowledge of capacity and performance characteristics of highways is fundamental to traffic engineering. Highway capacity concerns itself both with the ultimate carrying ability of various facilities and also with the relative service characteristics of the facilities operating at some fraction of the capacity volume. The study of highway capacity is both qualitative and quantitative, which permits evaluation of both the adequacy and the quality of the vehicle service being provided by the facility (Pignataro, 1973). The deficiencies of an existing highway system can be evaluated by comparing demand volumes to the capacity of the existing facilities. The effectiveness of changes in the geometrics of the highway system can also be evaluated with respect to capacity. Design of a new facility is based on the capacity analyses coupled with the projected traffic demand. Thus traffic demand and capacity analyses are two major aspects of highway transportation system planning.

1.1.1 Highway Capacity

Highway capacity is defined as the maximum number of vehicles that can pass over a given section of lane or roadway during a given time period (mostly an hour), under prevailing roadway and traffic conditions (Pignataro, 1973). It is a common practice to express capacity in terms of passenger car units (PCUS). Level of service is associated with different operating conditions that occur on a facility when it accomodates various traffic volumes. It is a quantitative measure of the effect of a number of factors that include speed and travel time, traffic interruptions, freedom to manoeuvre, drivers comfort and convenience etc. Six levels of service, designated 'A to F', have been defined in the Highway Capacity Manual (HRB, 1965). The capacity of various highways have also been defined for each of the six levels of service.

In India, highway capacity has often been expressed as the maximum daily traffic volume that can operate on the highway. The daily capacities of single and double lane highways were considered to be 1000 and 4000 PCUS respectively (Sehgal, 1967). Very limited field studies on highway capacity seem to have been done in India.

Eastern Region Transport Survey Group (1966) has recommended a daily capacity of 3000 PCUS for single lane and 10000 PCUS for two lane roads. A study team of National Council of Applied Economic Research (NCAER) has suggested that single and double lane roads could carry 3000 and 7500 PCUS respectively per day (NCAER, 1965). The factors for reducing capacity to compensate for pavement width and lateral obstructions below ideal conditions have also been specified by Eastern Region Transport Survey Group (1966).

1.1.2 Traffic Demand

Estimation of future traffic demand is necessary for designing the transportation facilities. Future traffic demand is generally estimated by a three step procedure: (i) Trip generation forecasting; (ii) Distribution of these trips between zones, and (iii) Assignment of these trips to a future network (Martin et al., 1965). Trips generated in a region are dependent upon a number of variables like population, income of people, number of vehicles, agricultural and industrial production etc. (Roberts, 1966). Based on present and past traffic data, the structure of trip generation model is established and used to obtain an estimate of future traffic for some target periods. Traffic demand levels are not deterministic, but are probabilistic.

Furthermore, the annual forecasted traffic for a design year is not constant throughout the year, but there exist seasonal cyclic variations within the year. The major variations may be expressed by monthly, weekly and daily time patterns of the traffic flow (HRB, 1965). Peaking characteristics are observed even within the peak hour. The entire phenomenon of traffic flow volume is thus time dependent and probabilistic.

Traffic facilities are usually designed on the basis of Design Hourly Volume (DHV). Due to the extreme variations in traffic flow during the day and throughout the year, it was noted in the capacity manual (USDC, 1950) that:

> .... If a roadway facility is to be so designed that traffic will be properly served, consideration must be given to the brief but frequently repeated rush-hour periods. It is neither wise nor economical, however, to provide for the extreme hourly volumes of traffic that may occur but a few times during a year. The law of diminishing returns must be applied to fix the highest hourly volumes which will justify the necessary expenditure of funds to provide the added capacity.

The hourly volumes of traffic may be expressed as a percentage of Annual Average Daily Traffic (AADT). The cumulative frequency distribution of hourly traffic indicates that at about the thirtieth highest hour, the slope of curve changes rapidly and it may not be economical

to design for a volume higher than this volume. Where data are available, or for major locations, DHV may be derived from an actual analysis of data. Where detailed data are not available, data for similar highways may be used to determine the DHV by expressing it as a percentage of AADT (Wohl and Martin, 1967). The DHV for the highway under consideration is obtained by multiplying this percentage by AADT.

1.2 Statement of the Problem

Traffic flow on a highway is a stochastic process which may consist of deterministic components including trends and cycles, and probabilistic components. Stochastic modelling deals with representing these components in analytic terms. Time series analysis considers the sequence of values of the variable in time (Box and Jenkins, 1970). Several time series models are available and can be used for representing the temporal variations of traffic flow. Different categories of vehicles generally have different temporal variations. It is proposed to develop time series models to represent monthly, daily and hourly variations of different categories of vehicles in the traffic flow. These models may be used for forecasting future traffic demand.

Traffic generally is not homogeneous, but consists of different categories of vehicles. In developing countries like India, it includes slow moving (man driven and animal driven) and fast moving vehicles among which there are wide variations in speed. It however seems necessary to represent the characteristics of each type of vehicle in terms of a common unit preferably the passenger car. They are referred to as Passenger Car Equivalents(PCES). The mixed traffic flow process is complicated because of some of the following factors: (i) It is a complex stochastic process; (ii) Free speeds of vehicles are not constant but are probabilistic; and (iii) There are interactions between vehicles moving in the same direction and also with vehicles moving in the opposite direction. These include acceleration, deceleration, overtaking etc.

In India, PCES are generally considered to be constants and to be independent of composition and volume level of traffic. Because of the complex nature of the process, it seems reasonable to assume that interactions are nonlinear and hence PCES are functions of volume and composition of traffic. It is necessary to investigate the interaction between slow and fast moving vehicles in the mixed traffic flow. It seems difficult, if not impossible,

to formulate and analytically solve for the interactions particularly because of the complex logical and probabilistic characteristics of the process. It is however possible to simulate the process in a digital computer, study the process and infer about the complex interactions.

The problem may hence be stated as follows:

(i) To represent monthly, daily and hourly variations of traffic using time series models; and

(ii) To study the interactions between slow and fast moving vehicles in the mixed traffic flow using computer simulation and hence to derive PCES of different categories of vehicles as a function of traffic composition and volume level.

1.3 Objectives of the Study

The following are the main objectives of the study:

(i) To develop stochastic models for the monthly and daily goods traffic on five highways touching Kanpur and to use these models for traffic forecasts;

(ii) Data concerning characterisation of traffic including probabilistic distribution of flow, free speed, operating speed and delays etc., are not available. Field studies were conducted to formulate appropriate component models;

(iii) To formulate and validate a simulation model for the mixed traffic flow on a two lane highway;

(iv) To simulate the mixed traffic process under varying volume levels and compositions to study the interactions; and

(v) To derive the PCES for each category of vehicle as a function of traffic volume and composition.

1.4 Scope of the Study

The subject matter under study is very broad. Furthermore, there are limitations of availability of data and computer time. So the present study was restricted to the following:

(i) Univariate time series models are developed for the monthly traffic data over the years and daily traffic data over the months. They are limited to the considerations of three categories of goods carriers approaching Kanpur on five different roads.

(ii) Simulation of the mixed traffic flow was carried out only for a two lane highway.

(iii) Simulation was done only for straight sections and effect of various geometrics like grade, curves and lateral obstructions, etc., were not studied.

(iv) Free speed distribution parameters of different categories of vehicles used in this study were as estimated from field observations on one road only.

(v) Traffic volume was taken to be equal in each direction. The effect of different directional distributions on speed volume relationships could not be studied.

1.5 Organisation of the Report

The study is reported in the following sequence:

(i) Problems of mixed traffic phenomenon are reviewed. Traffic field studies and analysis of data for characterisation of mixed traffic flow are presented (Chapter 2).

(ii) Time series models are presented and used to represent monthly, daily and hourly traffic variations. Time series models of monthly flows are also used for forecasting (Chapter 3).

(iii) Problems in mathematical modelling of mixed traffic flow are considered. Because of limitations of analytic models, the necessity of computer simulation of the process is considered (Chapter 4).

(iv) Computer simulation model for the process is formulated and validated using available field data (Chapter 5).

(v) Simulation experiments for different compositions and volume levels are planned. The simulation results are analysed to evaluate the interactions between the vehicles. The complex interactions are represented in terms of PCES for any catagory of vehicle as a function of traffic volume and composition. A nonlinear conditional multiplicative model is derived to represent the interactions (Chapter 6).

(vi) Study is summarised, conclusions are drawn and suggestions are made for future investigations (Chapter 7).

2. CHARACTERISATION OF MIXED TRAFFIC FLOW

2.1 General

The ability of a roadway to accommodate traffic depends to a large extent on the physical features of the roadway. However, there are other factors not directly related to the roadway features which are also of major importance in determining the capacity. Most of these factors relate to the variations in traffic demand and the interaction of vehicles in the traffic stream (Pignataro, 1973).

2.1.1 Stream Characteristics

Three basic measures of stream characteristics are volume, speed and density as they together describe the quality of service. Speed-volume relationship for uninterrupted flow is shown in Fig. 2.1. As volume increases, the space mean speed of traffic decreases until a critical density is reached. Thereafter, because of jamming, both volume and speed decrease (Wohl and Martin, 1967). Fig. 2.2 shows that density increases with volume upto the point of critical density. Thereafter volume decreases as density continues to increase to a maximum value known as jam

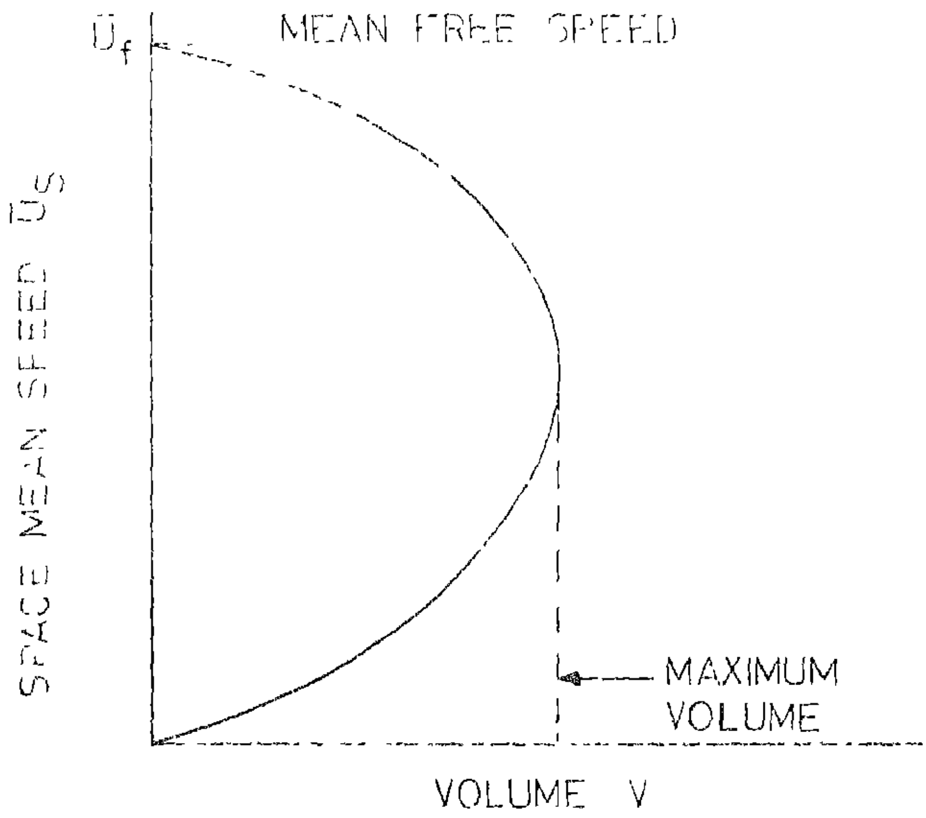


FIG. 2.1 SPEED VOLUME RELATIONSHIP

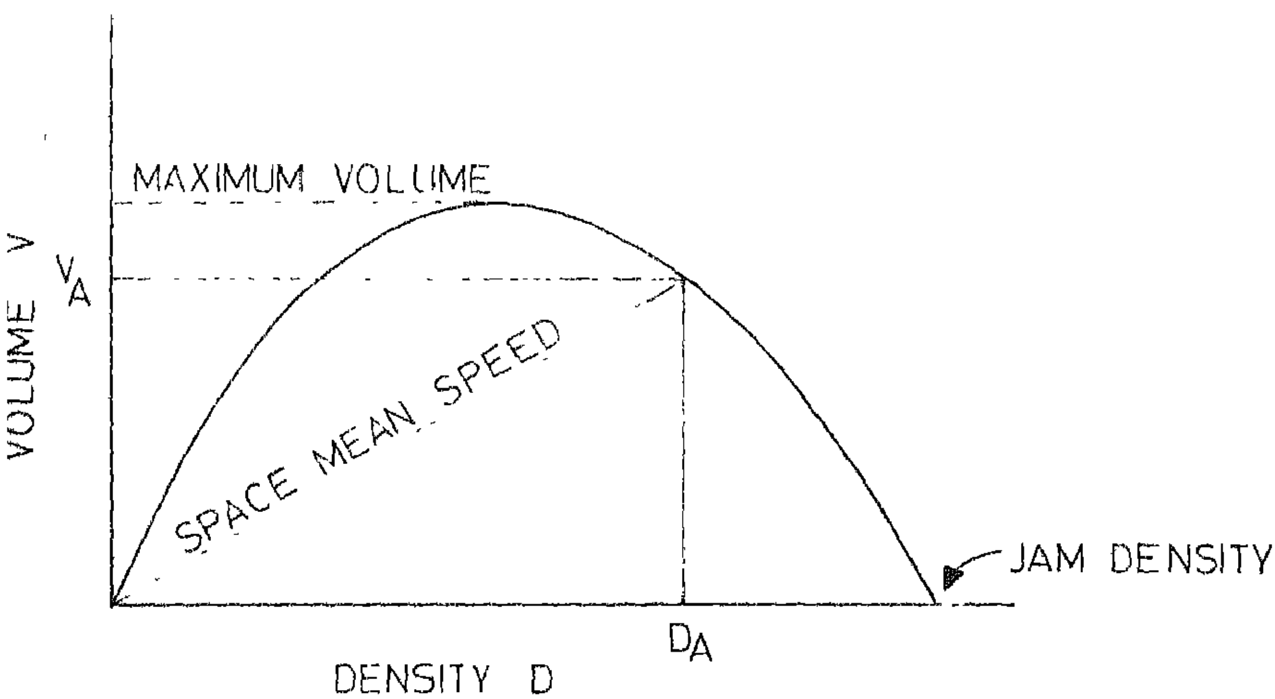


FIG. 2.2 VOLUME DENSITY RELATIONSHIP

density. Speed-density relationship in the upper range is similar to speed-volume relationship in that speed decreases with increasing density. Beyond the point of critical density, density continues to increase whereas the volume decreases.

2.1.2 Variations of Traffic Flow

Traffic flow is a stochastic variable characterised by the peak hourly rate. Traffic volume variations within the peak hour may seriously affect the stream characteristics. Capacity is normally defined in terms of vehicles per hour. If all the vehicles are equally spaced, determination of operating characteristics is simple. However, the vehicles do not move at uniform headways; rather they tend to form groups. Thus the rate of traffic flow for intervals of less than an hour can substantially exceed the peak hour rate. A peak hour factor that relates the peak interval and the peak hour volume, may be defined empirically (HRB, 1965). When demand exceeds capacity, congestion starts building up within the peak interval or extend over a longer period depending upon the traffic flow subsequent to the peak rate. Hence it is necessary either to provide excess capacity over the full hour to accomodate peak intervals

of flow or to permit limited congestion . For a rational analysis of the stream characteristics, the actual operating headway distribution is needed.

2.1.3 Passenger Car Equivalents(PCES)

PCES of vehicles, as specified by Central Road Research Institute (CRRI) and Ministry of Transport (MOT) of India, are given in Table 2.1 (Sehgal,1967). They show a wide difference in values. Furthermore, these PCES are constant and are considered to be independent of traffic volume and composition. Highway traffic at low volumes does not generally influence the speeds, and vehicles continue to move essentially at their free speeds. But as the volume increases, it is no longer possible for the vehicles to move at free speeds. When a fast moving vehicle is behind a slow moving vehicle immediately ahead of it, the fast moving vehicle desires to overtake. For a two lane road, the overtaking vehicle has to travel for some distance in the wrong lane (right lane in India) till the overtaking operation is completed. Overtaking is possible only if no vehicle coming from opposite direction comes in conflict with the overtaking vehicle. If overtaking cannot be carried out, then the overtaking vehicle has to reduce its speed, resulting in considerable delay to the fast moving vehicle.

TABLE 2.1 PASSENGER CAR EQUIVALENTS (PCES) FOR STRAIGHT SECTIONS OF ROAD

Type of Vehicle	Passenger Car Equivalents (PCES)	
	Central Road Research Institute(CRRI)	Ministry of Transport(MOT)
Car, Jeep, Van	1.0	1.0
Bus	3.6	3.0
Truck	2.7	3.0
4 Seater Rickshaw	0.7	-
2 Seater Rickshaw	0.6	-
Motor Cycle	0.3	1.0
Scooter, Autocycle	0.2	1.0
Bicycle	0.4	1.0
Tonga	2.6	6.0
Bullock Cart	10.7	6.0
Hand Cart	2.7	6.0
Cycle Rickshaw	1.4	1.0

With increases in volume level and proportion of slow moving vehicles in the mix, the number of gaps available in the opposite direction to allow overtaking decreases, resulting in considerable restriction on the overtaking operations. The operating speeds of the fast moving vehicles thus considerably get reduced. The flow characteristics of the mixed traffic flow are quite complex due to the interactions between the vehicles. Because of these complex interactions, PCES of different catagories of vehicles are considered to vary with traffic volume and composition.

2.2 Traffic Field Studies

2.2.1 Necessity

For characterising the mixed traffic flow, it is necessary to identify the probabilistic characteristics of the flow process and the effect of interactions between the vehicles. The following information is needed for system modelling (Naylor et al., 1966) :

(i) Characteristics of the input like traffic volume, composition, gap distribution , free speeds, etc.,

(ii) Characteristics of the system like physical features of the roadway and other control measures, if any; and

(iii) Characteristics of the output like density, travel time, delay time, operating speed etc., of various vehicles under different input conditions.

The accuracy of the system model depends upon the data available. Information concerning the above characteristics was not available from records and so it was necessary to conduct field studies.

2.2.2 Site Selection

Grand Trunk (G.T.) Road -NH_2, is a major link between the north and east of India with a considerable amount of traffic. A 2.25 km long section of this road, between Kalyanpur and Rawatpur, was selected for field studies. This section lies within Kanpur Metropolitan limits and thus has considerable amount of local fast and slow moving traffic alongwith the inter-city traffic. It had the following advantages with respect to the objectives of the study:

(i) The selected section has 6.0 metre wide cement concrete pavement with about 1.5 metre wide unpaved shoulders on either side. The pavement is in good condition for free flow;

(ii) The section lies on straight reach with no restrictions on sight distance. Thus the effect of curves,

sight distance etc. on operating speeds does not come into play.

(iii) There is no intersection on the selected section and so there is no cross movement affecting the flow of vehicles on the road;

(iv) The road profile is almost flat with the maximum gradient being even less than one percent. Running speeds of vehicles are thus not affected by gradients; and

(v) There is a railway track on one side of road and very little man made construction on other side. There is, therefore, little possibility of pedestrians crossing the road and delaying the vehicular movement.

The section is hence considered ideal for estimation of interaction effects and hence chosen for field studies.

2.2.3 Siting of Posts

Five observation posts were set up for data collection as shown in Fig. 2.3. Traffic volume of different categories of vehicles was recorded at 5 minute intervals at posts 1 and 5 respectively for incoming and outgoing traffic. Data for free speed of vehicles were recorded at posts 1, 2 and 3. Information concerning travel time, delay time , gap distributions, etc., were recorded at posts 1, 3, 4 and 5.

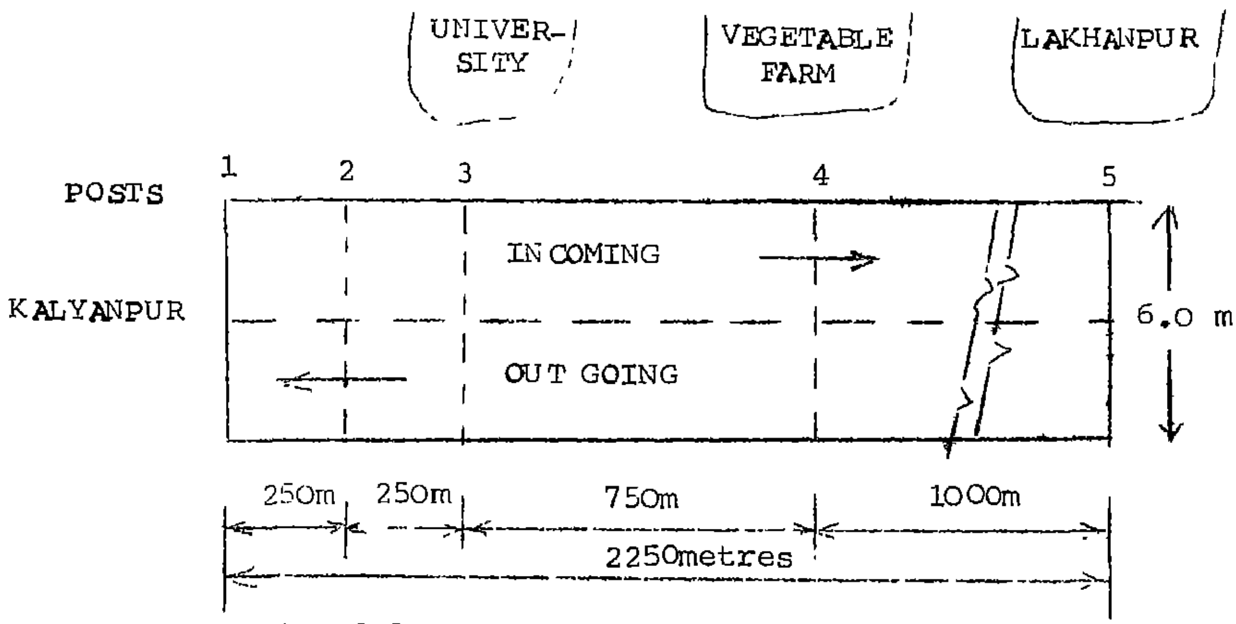


FIG. 2.3 LAYOUT OF ROAD SECTION SELECTED FOR FIELD STUDIES

2.2.4 Duration of Survey

Traffic survey was conducted round the clock for ten days in June, 1973. Twenty five engineering students were trained for proper collection of data. The study was conducted in four shifts per day. Traffic volume was recorded for all the four shifts. Free speed, gap distribution and travel time data were recorded for one shift on each day so as to cover all the four shifts twice during the period of study. On the whole, information for about 70,000 vehicles was recorded.

2.2.5 Data Collection

Stop watches were synchronised for each shift before starting the data collection. Procedures concerning the recording of data are briefly discussed.

(i) Traffic Volume Data: Incoming and outgoing volumes of different categories of goods and passenger carriers were recorded at 5 minute intervals. Volume of bicycle traffic was very high and this was recorded at 30 second intervals. For goods carriers, the volume of loaded and empty vehicles were separately recorded.

(ii) Free Speed Data:- Three posts namely 1, 2 and 3 were set up with the distance between successive posts as 0.25 km.(Fig.2.3). Free speed information was recorded by noting the time of crossing of a mark on the road by each vehicle at the three posts. In order to identify the vehicle, additional information including its category, viz., car, truck, bullock cart, bicycle,etc.; make , colour, etc., were also noted. The positions of the observers were such that they could easily see the movement of any vehicle over a 0.25 km section on either side of the posts. So the observers also noted whether the vehicle is not accelerating, decelerating or overtaking and hence is moving freely during the entire section of 0.5 km. If it is observed that there is

any restriction to free flow of a vehicle, information concerning that vehicle was crossed out from the records. If information about a vehicle was crossed even by one observer, then that vehicle was not considered for the free speed analysis. Scanning of vehicles was done from the type and identification recorded at each of the three posts. Travel time for free flow over 0.5 km distance was computed and free speed for each vehicle estimated.

(iii) Travel Time and Operating Speed Data: When a vehicle moving from either direction crossed a post (1, 3, 4 or 5) , information concerning the type of vehicle, its make, colour , time of crossing,etc., was recorded. Separate observers were stationed for the two directions. Knowing the time of entry into the section at post 1 (or 5), travel times were computed from observation records at posts 3, 4 and exit post 5 (or 1) . The data were also used for determination of the following:- (i) operating speed of vehicles; (ii) delay time of vehicles; (iii) density of different categories of vehicles in the section at any instant; and (iv) interarrival time gaps from either direction.

2.3 Frequency Analysis of Field Data

2.3.1 General

Field data were analysed to obtain the necessary inputs and outputs of the system model. Analysis was done for the following characteristics:

(i) Traffic volume distributions of various categories of vehicles at different time periods. This is discussed in Sec. 3.6.

(ii) Interarrival time gap distributions of the vehicles on the road.

(iii) Free speed distributions of different categories of vehicles.

(iv) Travel time, delay time and operating speed distributions of various vehicles at different volume levels and traffic composition. These deal with the output of the system and are useful for validating the system model (Sec. 5.10).

2.3.2 Interarrival Time Gaps

Vehicles arrive at the section from either direction, apparantly without any regularity. The flow may be considered as random, i.e., the number of vehicles arriving in any interval of time follow a probability distribution and is

independent of the number of vehicles that arrived during any previous time interval. From the arrival time data, interarrival time gaps were computed. The frequency distribution of arrival gaps is shown in Fig.2.4 for one shift of a day. Fig. 2.5 shows the cumulative distribution of arrival gaps. Assuming that the vehicle arrivals are randomly distributed, Poisson distribution may be used for the number of vehicles arriving in unit time and in turn exponential distribution may be used for the interarrival time gaps (Gerlough, 1964). Vehicles moving on a highway in the same direction require at least some physical spacing and hence a minimum time gap. Fig. 2.4 and 2.5 show that the minimum observed headway, h_{min} was 1 second. By shifting the exponential curve from the origin along the x-axis by an amount h_{min}= 1 second, the minimum headway, the following shifted exponential distribution was fitted to the interarrival time gaps.

$$\text{Prob}\ (h < t) = \left[1 - e^{-\frac{(t - h_{min})}{(\bar{t} - h_{min})}}\right] \quad \text{for } t \geq h_{min} \qquad (2.1)$$

$$\text{Freq}\ (h < t) = V\left[1 - e^{-\frac{(t - h_{min})}{(\bar{t} - h_{min})}}\right] \quad \text{for } t \geq h_{min} \qquad (2.2)$$

where h = interarrival time gap in seconds; $\bar{t}$ = T/V = average interarrival time gap in seconds; V = total number of vehicles arriving during time T; and h_{min} = 1 second.

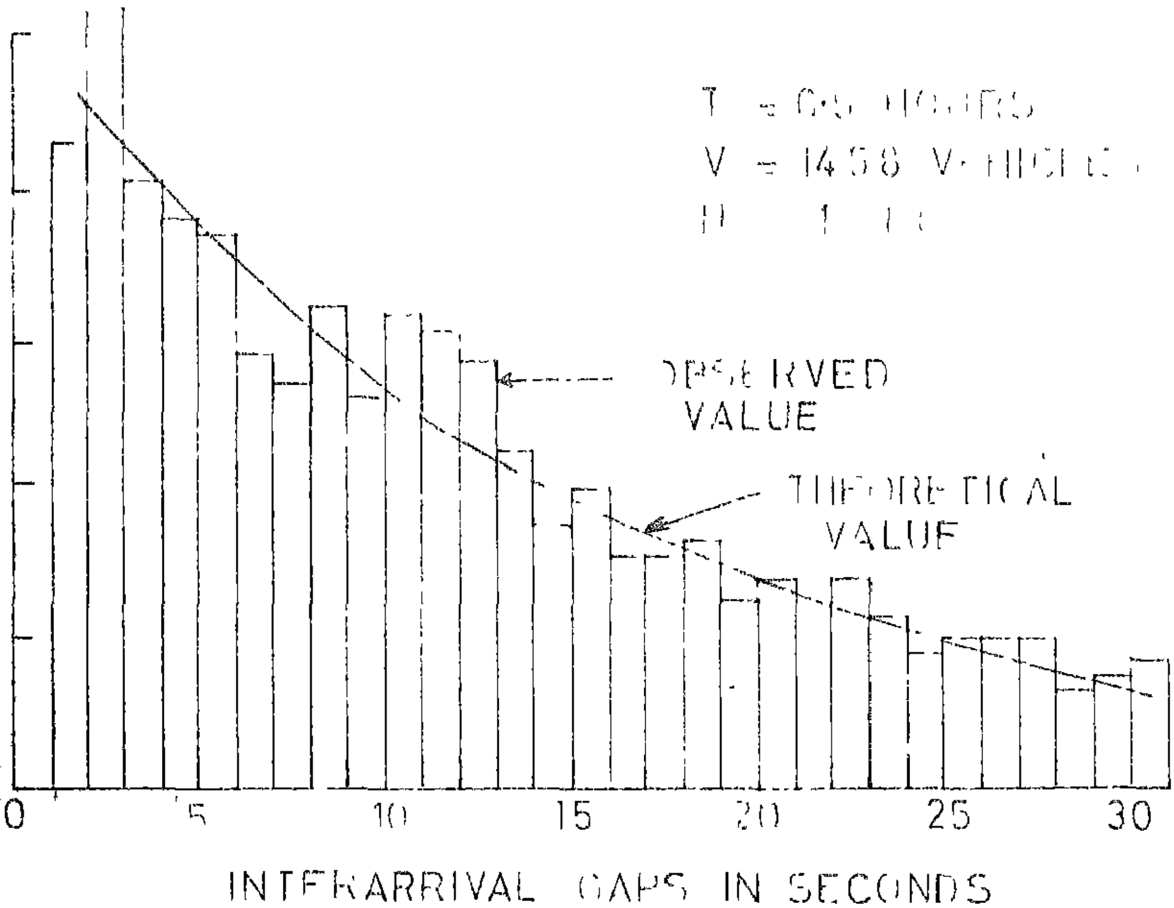


FIG. 2.4 FREQUENCY DISTRIBUTION OF INTERARRIVAL GAPS

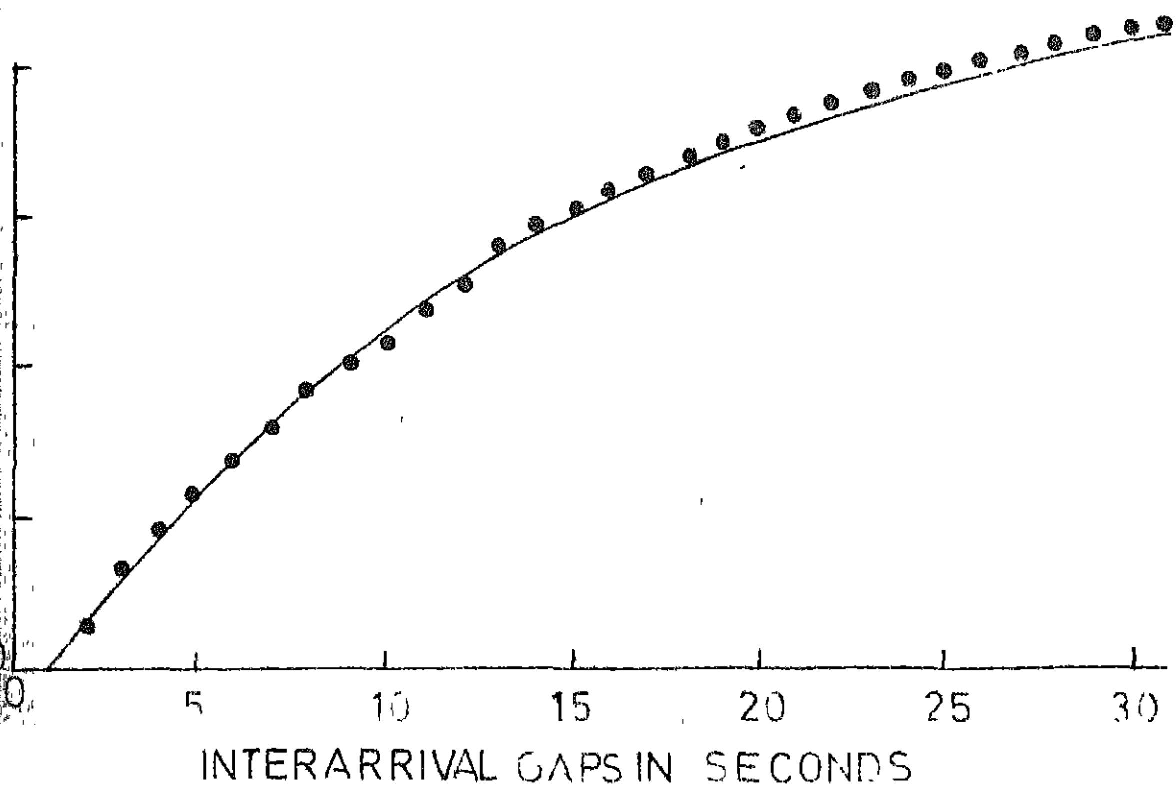


FIG. 2.5 CUMULATIVE FREQUENCY OF INTERARRIVAL GAPS

The theoretical frequencies of arrival gaps as obtained from shifted exponential distribution are also shown in Figs. 2.4 and 2.5 alongwith the observed frequencies. Goodness of fit was tested by Chi-square test. Results confirm that shifted exponential distribution fits well with the interarrival time data in other cases also.

2.3.3 Free Speed Distributions

The free speed of vehicles were computed from the times to travel 0.5 km. under free flow conditions. The vehicles were categorised into the following six types, each having similar characteristics:

(i) Heavy motor vehicles like trucks and buses;

(ii) Medium motor vehicles like passenger cars, jeeps, tempos and taxis etc.;

(iii) Light motor vehicles like scooters and motorcycles;

(iv) Horse driven vehicles like tongas or kharkhara;

(v) Bullock carts; and

(vi) Bicycles and cycle rickshaws.

Free speed data were recorded for about 2300 vehicles. They include all motor vehicles, tongas and bullock carts moving at free speed during the period of study and only

around 400 bicycles whose volume was much higher. The free speed of vehicles of a particular category were found to vary. The variations may depend upon the make of vehicle, its age , driver's characteristics, etc. The frequency of free speed distributions of 480 heavy motor vehicles is shown in Fig. 2.6. It has a peak in the middle with low values at very high and very low speeds. The observed mean was 44.6 kmph with a standard deviation of 6.65 kmph. A normal distribution was fitted to the field data and parameters were estimated.

For estimating the parameters, it is necessary to determine the probability density function $f(x)$ or the cumulative distribution function $F(x)$ from the observed data. $F(x)$ was determined by the plotting position formula (Benjamin and Cornell, 1970).

$$P(x) = \frac{m}{N + 1} \qquad (2.3)$$

where $P(x)$ is the exceedence probability $P(x) = 1 - F(x)$; m = rank of the observation with the highest value having $m = 1$; and N = total number of observations.

Plotting positions $P(x)$ vs. x were plotted on normal probability paper and a straight line was fitted by eye sight. The fitted straight line represents a normal distribution with a mean of 44.3 kmph and a standard

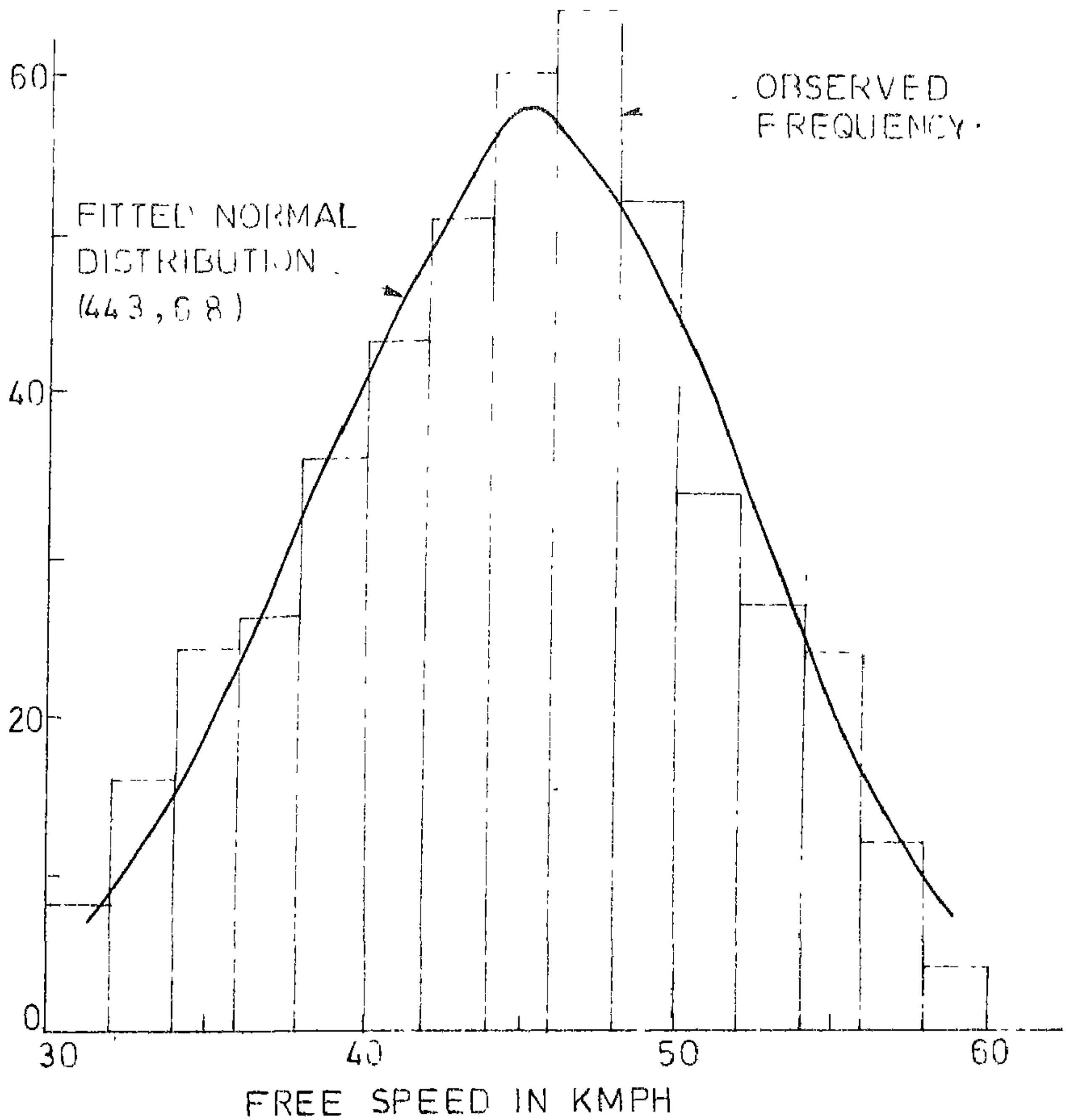


FIG. 2.6 FREQUENCY DISTRIBUTION OF FREE SPEEDS FOR HEAVY MOTOR VEHICLES

deviation of 6.8 kmph. Chi-square test indicated the goodness of fit at 10 percent significance level. Normal distribution was fitted to free speed of other categories of vehicles also and the estimated parameters are shown in Table 2.2. The maximum deviation of speeds from mean was found to be not more than thrice the standard deviation (sd) and so the normal distributions were censored at ($\bar{x} \pm 3$ sd) limits.

The results of frequency analysis are used in the validation of simulation model and in the simulation of mixed vehicular traffic.

TABLE 2.2 PARAMETERS OF NORMAL DISTRIBUTIONS FITTED TO FREE SPEED OF VEHICLES

S.No.	Type of Vehicle	Sample Size	Mean Free Speed kmph	Standard Deviation of Free Speed kmph
1.	Heavy Motor Vehicles ie. Trucks and Buses	480	44.30	6.8
2.	Medium Motor Vehicles ie. Cars, Jeeps, Tempos etc.	585	49.95	8.05
3.	Light Motor Vehicles ie. Scooters, Motocycles etc.	298	39.80	7.6
4.	Horse Driven Vehicles ie. Tonga, Kharkhara	354	12.8	1.5
5.	Bullock Carts	82	5.2	1.1
6.	Bicycles and Cycle Rickshaws	512	11.9	1.8

3. STOCHASTIC ANALYSIS OF TRAFFIC FLOW

3.1 Introduction

Traffic volume varies within the year with seasonal weekly and daily variations and they are to be considered in traffic forecast. The system should be designed for the peak traffic in a suitable period, say an hour, perhaps with a certain probability of meeting the demand. The design should be based on stochastic modelling and analysis of historical data for a sufficiently long period.

3.2 Stochastic Processes

3.2.1 General

When the future variations can be predicted uniquely on the basis of the present and past values of the characteristics, a process may be considered to be a deterministic process. Actually the variations of traffic volume are erratic and future volumes cannot be uniquely predicted. Predictions are then to be made only by using probabilistic models and in terms of the chance of occurrence of an event. Furthermore, the sequence of occurrence of events is important in the analysis of traffic flow. Mathematical models may be used

to describe the process and to analyse its characteristics. The theory of stochastic processes deals in analytic terms with phenomena which develop in time in accordance with laws of probability (Box and Jenkins, 1970).

3.2.2 Time Series

A time series is an ordered set of realisations of the process. Let Z (t) be a characteristic defined in the time domain $t \in T$ ie., $[\, Z(t) \;;\; t \in T \,]$. The characteristic may be defined as a continuous function of time and then it is referred to as a continuous time series (Ramaseshan, 1972). However, traffic volume data are given for discrete values of time as sampled or quantised data. The time series are then referred to as discrete time series. Traffic volume observations may be made at constant intervals of Δt; say $(t_o + \Delta t)$, $(t_o + 2 \;\Delta t)$, $(t_o + 3 \;\Delta t)$,....., $(t_o + N \;\Delta t)$. Then the N successive observations Z_1, Z_2 , Z_3,..., Z_N respectively constitute a discrete time series.

3.2.3 Components of a Time Series

A historical time series record may be considered to consist of (Kisiel, 1969):

(i) Deterministic components like trend and cycles which are purely functions of time;

(ii) Other nonrandom components like autoregressive(AR) and moving average (MA) components that represent the internal and external correlation respectively;

(iii) Pure random component; and

(iv) Catastrophic events, which are extremely rare events that have occured during the limited period of record.

The time series may thus be represented as:

$$Z(t) = f_1(t) + f_2(t) + e_t + m\delta(t - t_c) \qquad (3.1)$$

where f_1 = deterministic components like trend , cycles etc.; f_2= persistence component; e = pure random component; and m = magnitude of catastrophic event occuring at time t_c. Catastrophic events are generally ignored in the representation of time series models.

A time series is said to be stationary if the generating mechanism of the process is independent of time (Yevjevich , 1972) i.e. the deterministic component and the parameters of the random component (and its distribution) are all independent of time. A stochastic process, Z(t), is said to be stationary in a wide sense if it possesses finite second moment and covariance of Z(t) and Z(t + h)

depends only on h for all t $\in$ T. Nonstationary time series thus need to be transformed to stationary series for estimating the persistence components.

3.2.4 Stages in the Selection of a Model

The selection of a stochastic model from historical records requires an iterative approach and the steps generally used are indicated in Fig. 3.1 (Box and Jenkins, 1970).

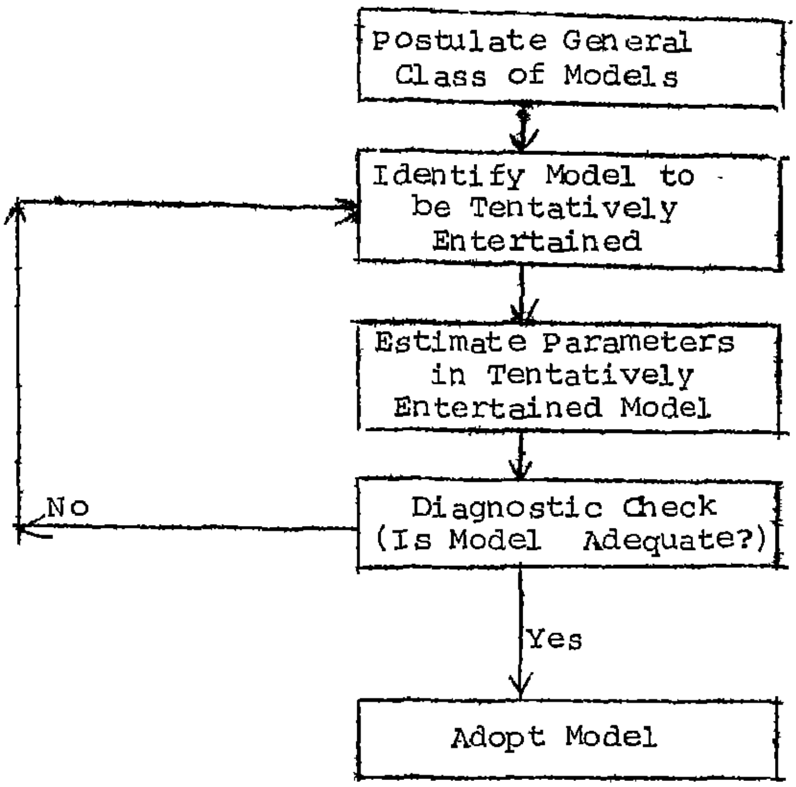


FIG. 3.1 ITERATIVE STAGES IN MODEL BUILDING

(i) A useful general class of models is considered from the interaction of theory and practice.

(ii) Rough methods for identifying subclass of the models are developed. The identification process can be used to yield rough preliminary estimates of the parameters in the model.

(iii) The tentative model is fitted to the data and its parameters are estimated. The rough estimates obtained during the identification stage may be used as starting values in an iterative method for estimating the parameters.

(iv) Diagnostic checks are applied with the object of uncovering possible lack of fit and diagnosing the cause. If any inadequacy is found, the iterative cycle of identification, estimation and diagnostic checking is repeated till a suitable representation is found.

3.3 Traffic Studies

Estimation of traffic demand on highways needs the following variations to be evaluated:

(i) Variations within the year and the month of the volume of different categories of vehicles using the road; and

(ii) Variations within the day to account for hourly volume characteristics.

For case (i) goods traffic data were available for five main roads approaching Kanpur (Subsec. 3.3.2) whereas for case (ii) field studies were conducted for data collection (Sec. 2.2).

3.3.1 Choice of Region

Kanpur is the largest industrial city in the state of Uttar Pradesh (U.P.). The population of Kanpur Metropolis, as per 1971 census, is 12.65 lakhs and it enjoys the eighth position among the metropolitan cities of India (KDA, 1975). It is served by two National Highways, G.T. Road (NH_2) and Lucknow-Kalpi (NH_{25}) Road. Thus it is connected to Delhi, Allahabad, Lucknow and Jhansi and thence to all the major cities of India, Hamirpur Road, a State Highway connects Kanpur to the districts of southern U.P. It is also connected to all the major cities of India by the broad gauge Central and Northern Railway lines and metre gauge North-Eastern Railway lines (KTC, 1975). Fig. 3.2 shows the transportation network in Kanpur Metropolis.

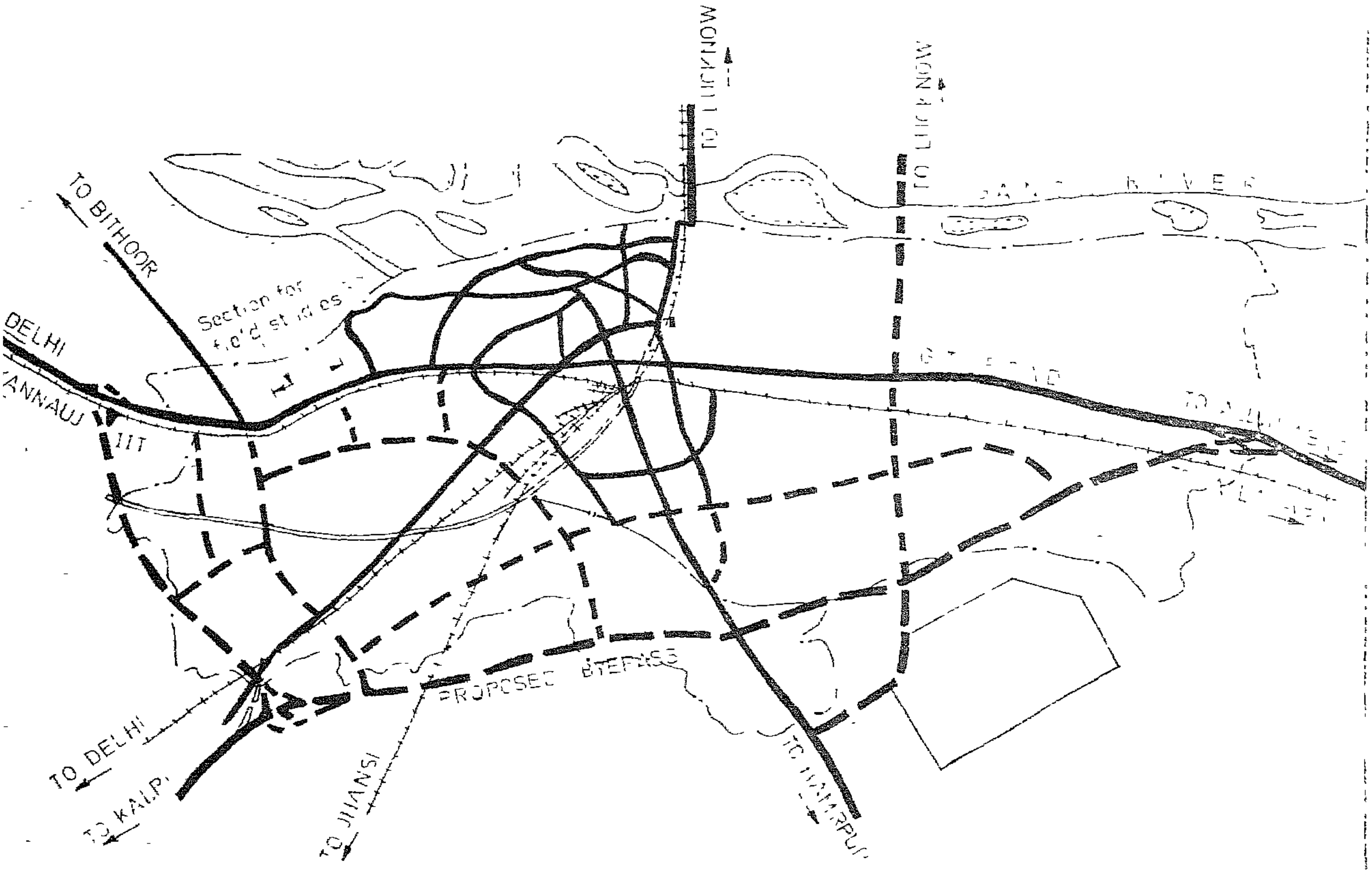


FIG 32 TRANSPORTATION NETWORK OF KANPUR METROPOLIS

3.3.2 Classification of Data

There is considerable road traffic between Kanpur and other adjoining cities due to various industries and commercial activities located at Kanpur. The agricultural produce of adjoining areas is also brought to Kanpur mostly by bullock carts. Further, the direct traffic between Delhi and Calcutta, and between Western U.P. and Bombay side, also pass through Kanpur on the two National Highways.

Delhi Road, Allahabad Road, Lucknow Road, Kalpi Road and Hamirpur Road are the five main road sections connecting Kanpur. Historical traffic data on these five roads were taken from the octroi records of Kanpur Municipal Corporation. The data were available only for goods traffic approaching Kanpur, which has to pay octroi. Daily data, recorded in shifts of eight hour duration, were available for seven years (April, 1966-March, 1973) for motor vehicular traffic. Bullock cart traffic data were available for three years. (April, 1970-March, 1973). The available data were classified into three categories, viz., (i) transit truck traffic i.e., commercial vehicular traffic touching Kanpur, but destined to some other place beyond Kanpur; (ii) incoming truck traffic i.e., commercial vehicular traffic destined for Kanpur; and (iii) incoming bullock cart traffic that

comes from adjoining villages to Kanpur.

Traffic volume studies were conducted on G.T. Road (Sec. 2.2) so as to analyse the hourly variations of all categories of vehicles in both directions.

3.3.3 Analysis of Data

Historical daily data of goods traffic and hourly data from field studies were used to develop and validate appropriate stochastic models. They include:

(i) stochastic models for monthly traffic flow for each of the three categories of vehicles on all the five roads;

(ii) stochastic models for daily traffic within a month in two roads for three representative months in a year; and

(iv) distribution of hourly traffic over a day for different categories of vehicles on G.T. Road.

3.4 Analysis of Monthly Traffic Flows

3.4.1 Introduction

Monthly traffic volumes, represented by the mean daily traffic of the month, were plotted for all the series, viz., incoming truck and bullock cart traffic on Hamirpur

Road and transit traffic also in case of other four roads. Data for transit truck traffic on Delhi Road (1966-1973) are shown in Fig. 3.3. The series were analysed using two different stochastic models, viz., the general time series model(Eq. 3.1), and a seasonal multiplicative Autoregressive Integrated Moving Average (ARIMA) model to be discussed later in Subsec. 3.4.3.

3.4.2 General Time Series Model

Trend Analysis: It deals with the identification and evaluation of long term trends, if any, in the time series. Fig. 3.3 shows that transit traffic on Delhi road increases generally with time, indicating the presence of periodic and random fluctuations. In some of the other series, annual traffic has more than doubled over a period of seven years, indicating a significant trend component.

Due to the presence of periodic and random fluctuations, it is difficult to identify the actual nature of trend and its parameters. Method of moving averages (Kendall and Stuart, 1966) was used for fitting the trend line. Let the weighted average ,

$$Z'(t) = \sum_{j=-m/2}^{j=m/2} b_j \, Z(t+j) \qquad (3.2)$$

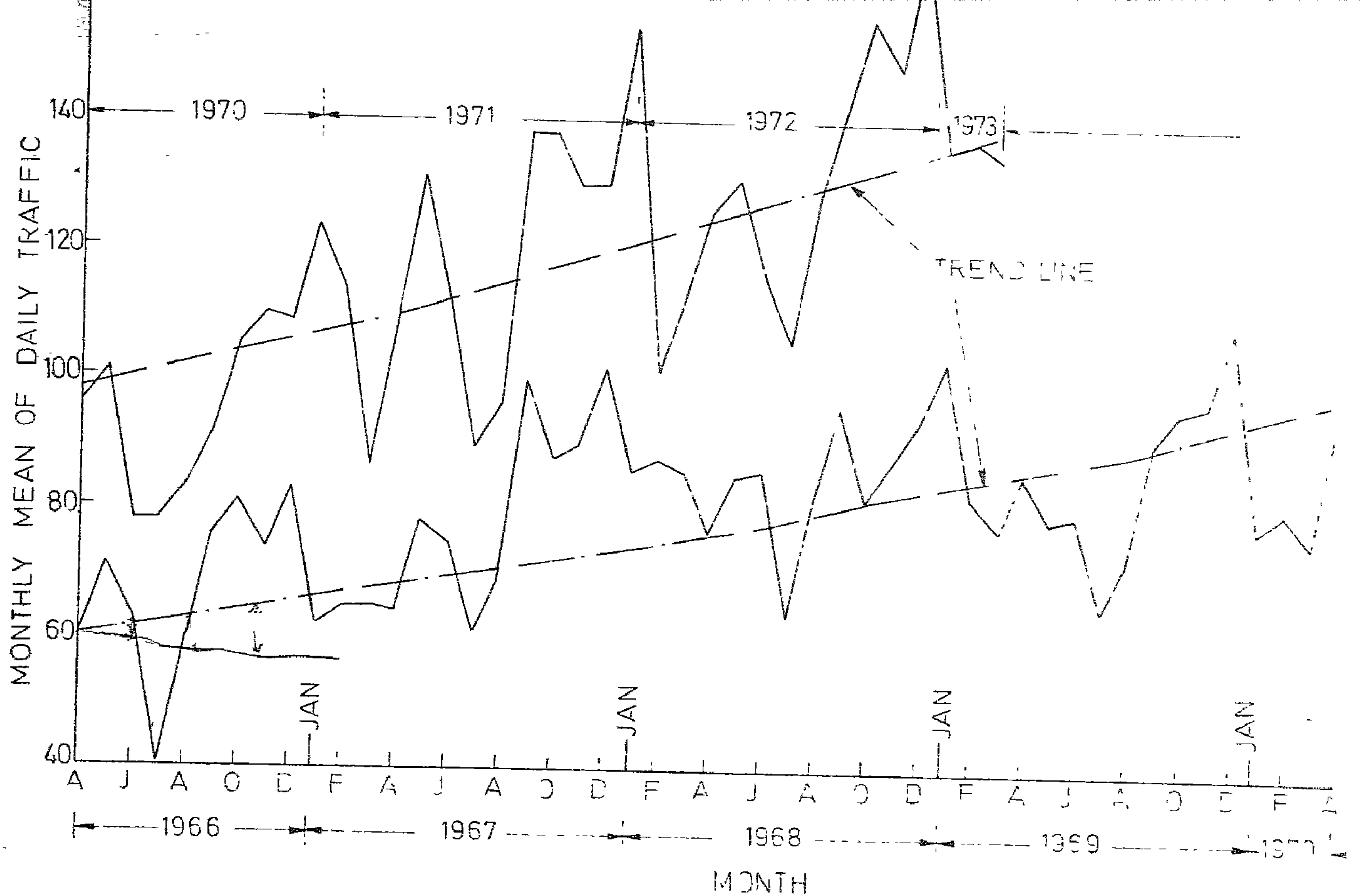


FIG. 3.3 MONTHLY MEAN OF DAILY TRANSIT TRUCK TRAFFIC (DELHI ROAD)

with $j = -\frac{m}{2}, -\frac{m}{2}+1, \ldots\ldots -1, 0, 1, 2, \ldots\ldots,$

$\frac{m}{2}-1, \frac{m}{2}$; and the

weights $b_j = \frac{1}{m+1}$

For the monthly traffic flows, 12 monthly moving average was used as it eliminates the annual cycle or its subharmonics, if any. The nature of $Z'(t)$ series was mostly exponential though linear trend was also observed for two series. The relationships were of the following types;

$$\left.\begin{aligned} Z'(t) &= A\, e^{Bt} \\ Z'(t) &= At^{B} \\ Z'(t) &= A + Bt \end{aligned}\right\} \qquad (3.3)$$

The regression coefficients A and B were estimated by the method of least squares for various time series and are shown in Table 3.1. The results indicate that generally bullock cart traffic increases at a faster rate than the motor vehicular traffic. The trend free series $Z_1(t)$ were obtained after subtracting the trend component from the original values.

Periodicity Analysis: Seasonal fluctuations of traffic volumes are indicated by the plots of the time

TABLE 3.1 REGRESSION ANALYSIS OF TREND FREE SERIES

Road	Category of Traffic	Trend Relationship	Regression Coefficients A	B
Delhi Road	Transit Truck	$Z'(t) = Ae^{Bt}$	60.0	0.012
	Incoming Truck	$Z'(t) = At^{B}$	88.0	0.170
	Bullock Cart	$Z'(t) = A + Bt$	30.0	2.500
Allahabad Road	Transit Truck	$Z'(t) = Ae^{Bt}$	61.0	0.020
	Incoming Truck	$Z'(t) = A + Bt$	75.0	0.210
	Bullock Cart	$Z'(t) = At^{B}$	36.0	0.350
Lucknow Road	Transit Truck	$Z'(t) = At^{B}$	42.6	0.0593
	Incoming Truck	$Z'(t) = At^{B}$	138.0	0.0782
	Bullock Cart	$Z'(t) = Ae^{Bt}$	149.0	0.01543
Kalpi Road	Transit Truck	$Z'(t) = Ae^{Bt}$	25.5	0.0207
	Incoming Truck	$Z'(t) = Ae^{Bt}$	68.0	0.017
	Bullock Cark	$Z'(t) = At^{B}$	34.0	0.430
Hamirpur Road	Incoming Truck	No Significant Trend	0.0	0.0
	Bullock Cark	$Z'(t) = At^{B}$	30.0	0.460

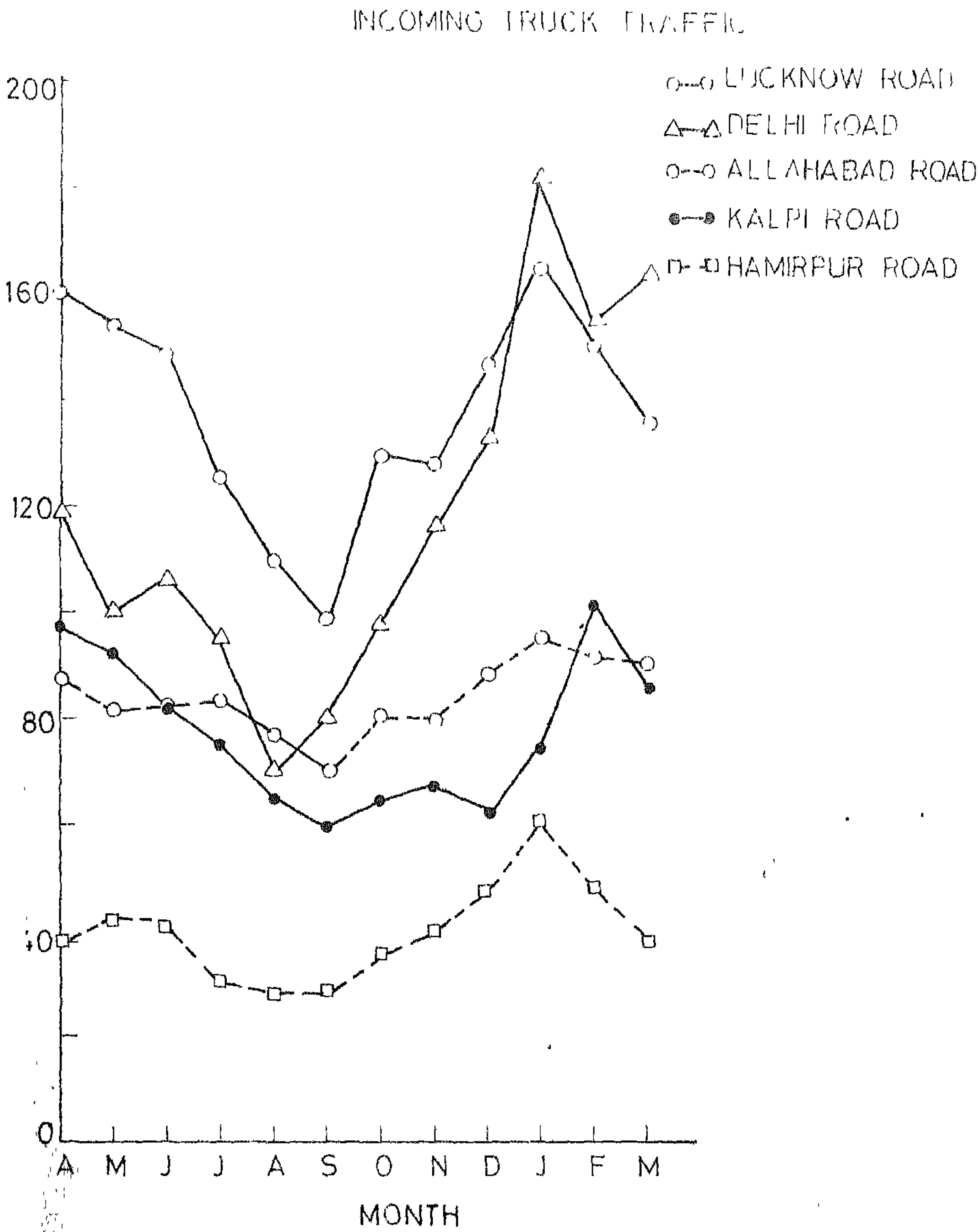
series. Hence the monthly mean of trend free traffic volumes were calculated and are shown in Figs. 3.4 and 3.5 respectively for incoming truck and bullock cart traffic. They indicate the seasonal variations of the means with a peak volume around January and the minimum around August-September for truck traffic. For bullock cart traffic, there are generally two peaks, the larger one around January and the other around June. Hence seasonal cycles seem to be present. It may be possible to relate the seasonal variations to agricultural operations and monsoon season.

The trend free monthly traffic flow series were found to have a skewed distribution. A log normal distribution was fitted to the monthly data. Let $X_1(t)$ denote the natural logarithm of the trend free series $Z_1(t)$ ie., $X_1(t) = \text{Log}_e\ Z_1(t)$. The periodic component of a time series can be represented by a sum of a number of harmonic components (Kisiel, 1969) ie.,

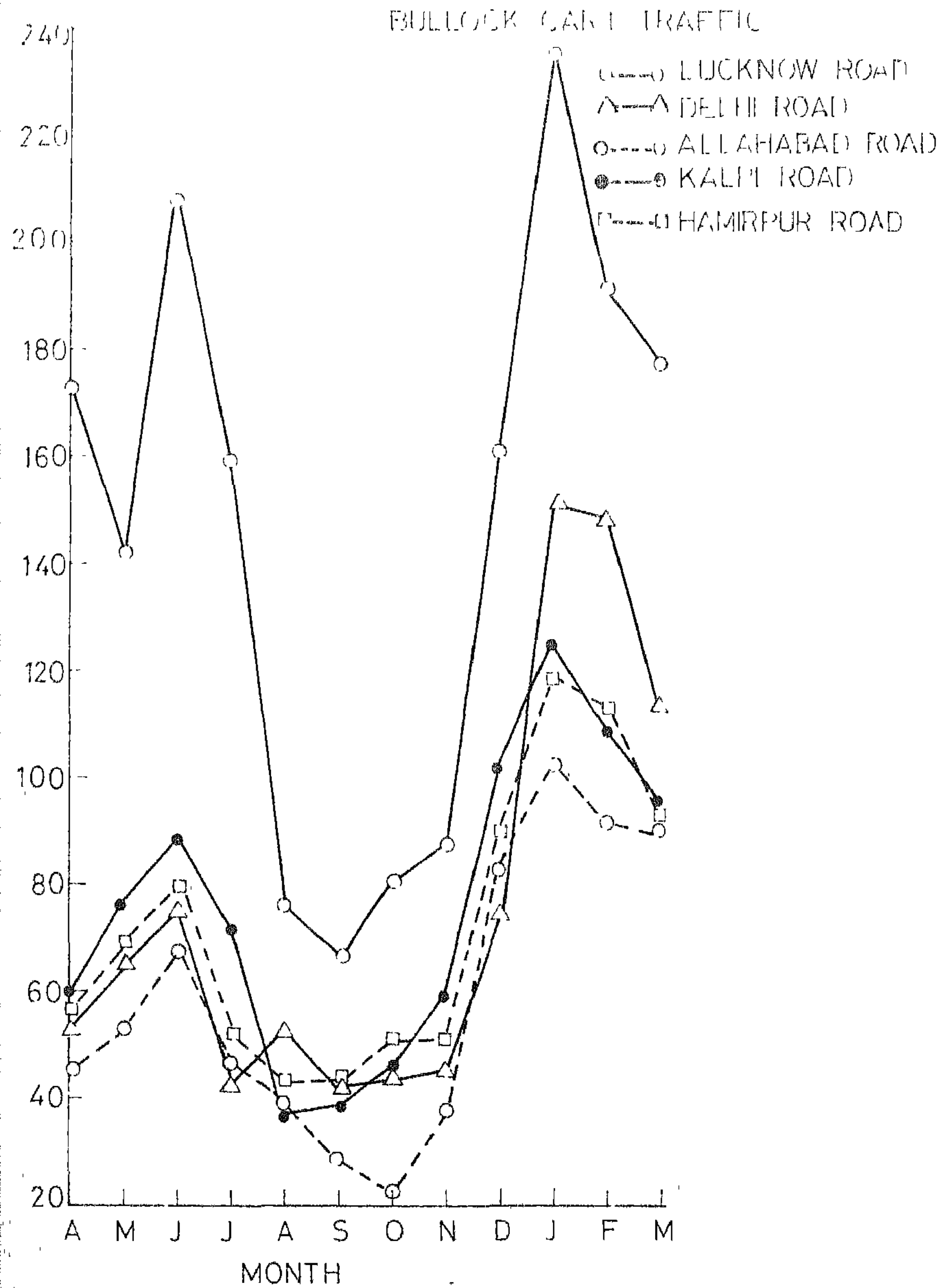
$$X_1(t) = \frac{a_o}{2} + \sum_{j=1}^{m} C_j \text{ Cos } \left(\frac{2\pi jt}{T} + \alpha_j\right) \tag{3.4}$$

or

$$X_1(t) = \frac{a_o}{2} + \sum_{j=1}^{m} A_j \text{ Cos } \frac{2\pi jt}{T} + \sum_{j=1}^{m} B_j \text{ Sin } \frac{2\pi jt}{T} \tag{3.5}$$



4 MONTHLY MEAN TRAFFIC OF TREND FREE SERIES



G 3·5 MONTHLY MEAN TRAFFIC OF TREND FREE SERIES

where $\frac{a_o}{2}$ is the mean of $X_1(t)$; T is the fundamental period; fundamental frequency $f_1 = \frac{1}{T}$; A_j and B_j are the Fourier coefficients; C_j and α_j are the amplitude and phase angle respectively of the j^{th} harmonic.

$$A_j = \frac{2}{N} \sum_{t=1}^{N} X_1(t) \cos \frac{2 \pi j t}{T} \tag{3.6}$$

$$B_j = \frac{2}{N} \sum_{t=1}^{N} X_1(t) \sin \frac{2 \pi j t}{T} \tag{3.7}$$

$$C_j^2 = A_j^2 + B_j^2 \tag{3.8}$$

and $$\alpha_j = \text{arc tan } \left(- \frac{B_j}{A_j} \right) \tag{3.9}$$

The periodic and persistence components may be identified through (i) correlogram (Box and Jenkins, 1970) which is a plot of serial correlation coefficients of the time series for different lags; (ii) partial autocorrelation coefficients; and (iii) power spectra which indicates the distribution of variance of the process among the different frequencies (Jenkins and Watts, 1968). Standard procedures were used in the estimation of the above functions.

The correlogram and spectra were computed for all the trend free traffic series. Some of the results are shown in Figs. 3.6 and 3.7 alongwith the 95 percent

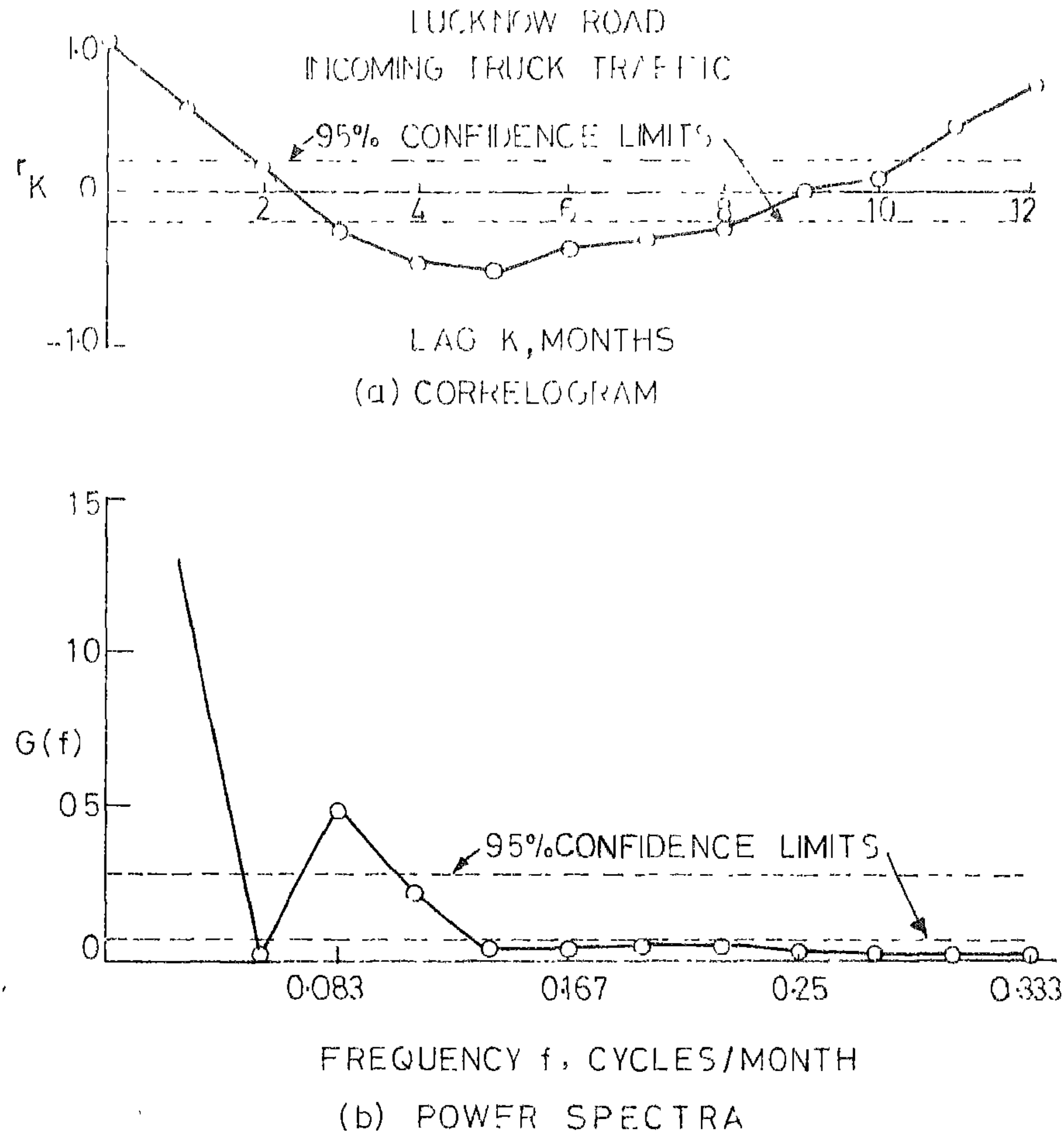
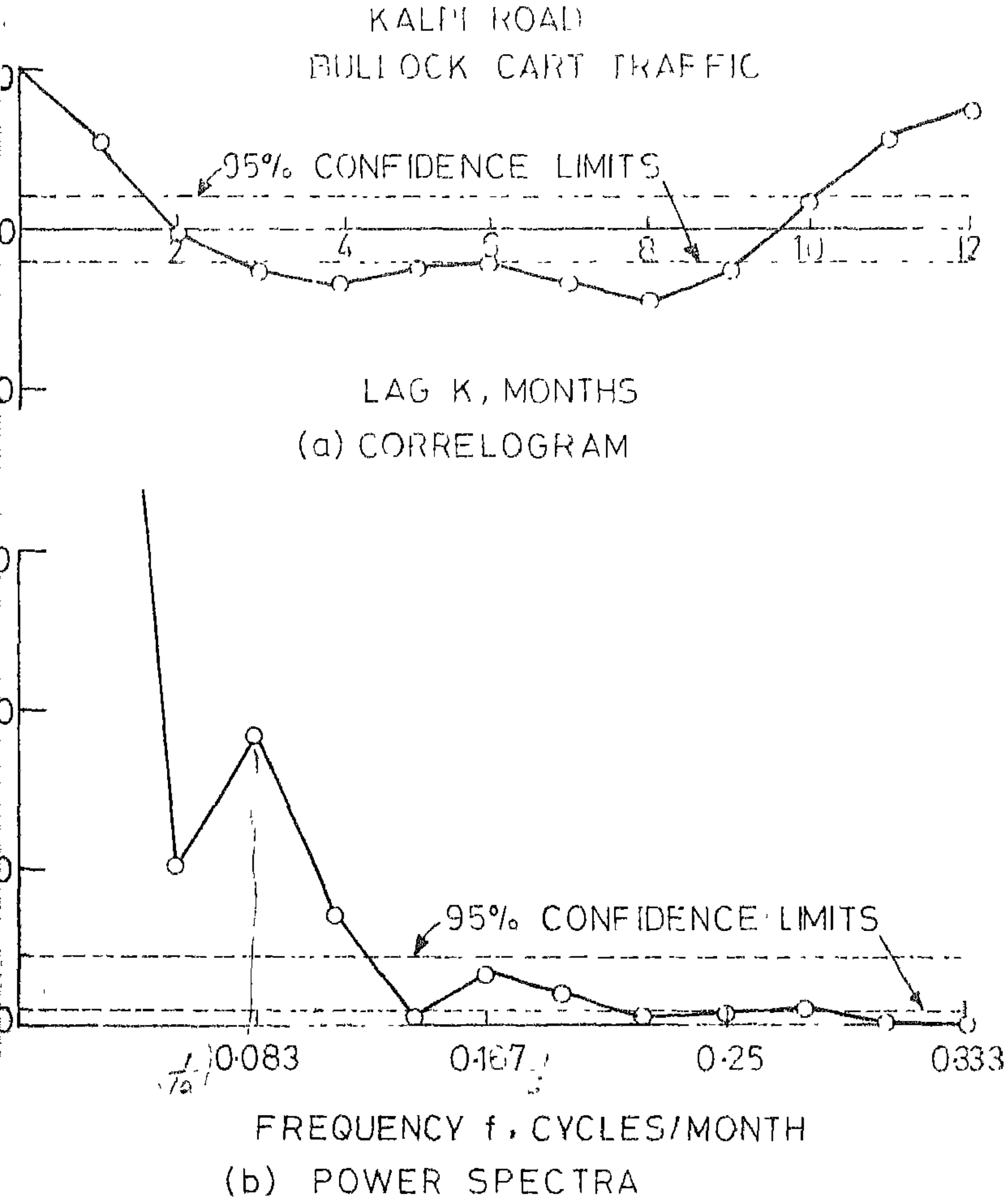


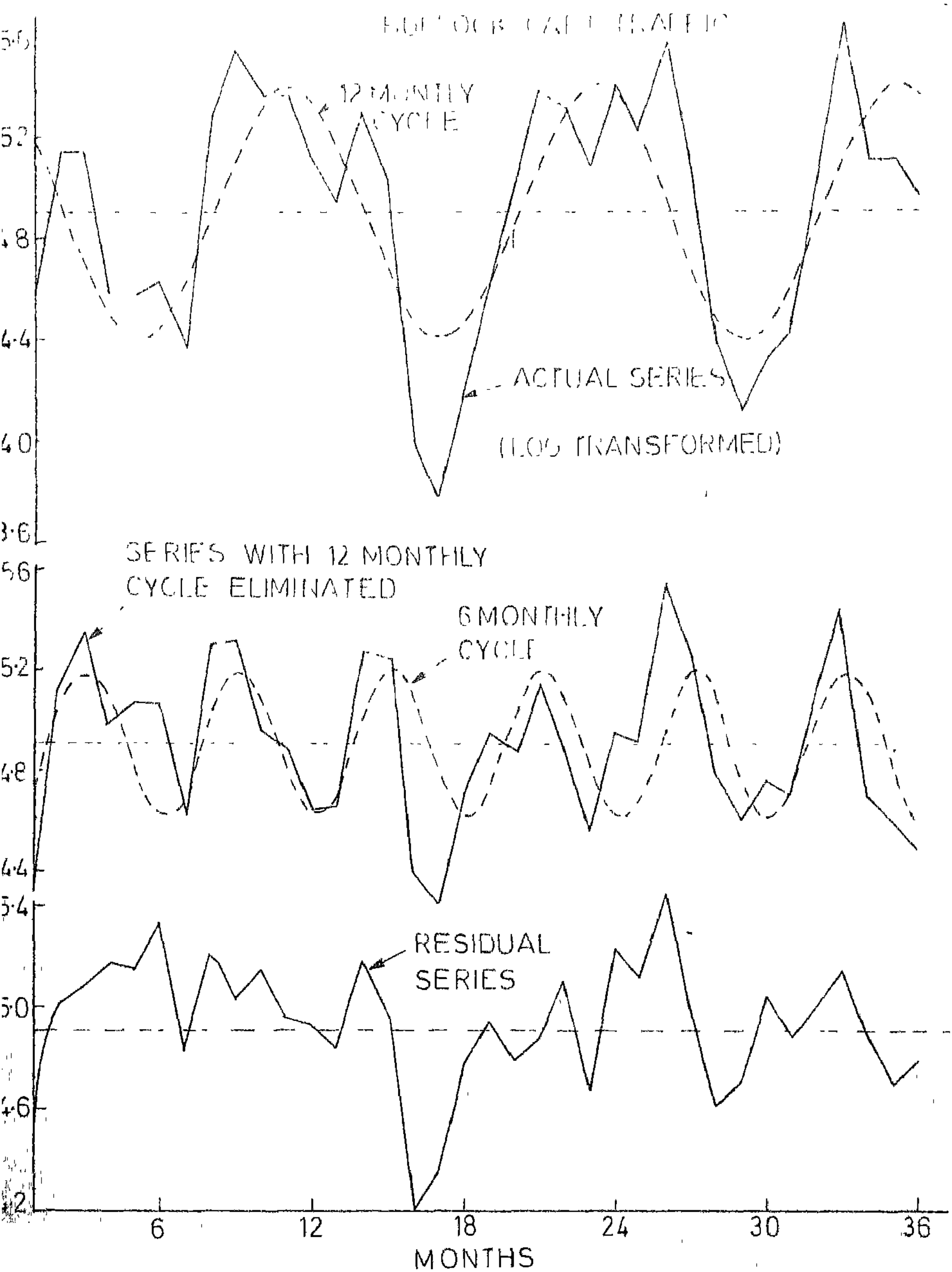
FIG. 3·6 CORRELOGRAM AND POWER SPECTRA OF TREND FREE LOG TRANSFORMED SERIES



·7 CORRELOGRAM AND POWER SPECTRA OF TREND FREE LOG TRANSFORMED SERIES

confidence level for a pure random series. Correlogram and power spectra analyses indicate the presence of a 12 monthly cycle for all categories of traffic (vide spikes at f = 0.0833 in Figs. 3.6 and 3.7). For the bullock cart traffic, 6 monthly cycle is also indicated (vide spike at f = 0.1667 in Fig. 3.7). The harmonic coefficients A_1 , B_1 , C_1 and α_1 for the 12 monthly cycle were estimated for each of the series and were eliminated from the $X_1(t)$ series. The residual series were again subjected to correlogram and spectral analyses. Six monthly cycles were found to be significant for the bullock cart traffic only. The harmonic coefficients A_2, B_2, C_2 and α_2 for the 6 monthly cycle were also estimated. Fig. 3.8 shows the $X_1(t)$ series with the 12 monthly cycle superimposed; the residual series (after elimination of 12 monthly cycle) with 6 monthly cycle superimposed, and the residual series after elimination of 6 monthly cycle as well. The parameters of the harmonic coefficients of various series are shown in Table 3.2.

Persistence Components: Persistence refers to the linkage or relationship existing between the value of a series at a given time with the earlier values, viz.,



3·8 HARMONIC COMPONENTS AND RESIDUALS OF TRANSFORMED SERIES

TABLE 3.2 HARMONIC COEFFICIENTS OF PERIODIC CYCLES

Road	Category of Traffic	Annual Cycle		Six Monthly Cycle	
		A_1	B_1	A_2	B_2
Delhi Road	Transit Truck	0.164	-0.034	-	-
	Incoming Truck	0.273	-0.136	-	-
	Bullock Cart	0.462	-0.207	-0.219	-0.0475
Allahabad Road	Transit Truck	0.133	-0.027	-	-
	Incoming Truck	0.098	-0.044	-	-
	Bullock Cart	0.275	-0.121	-0.145	-0.032
Lucknow Road	Transit Truck	0.145	-0.036	-	-
	Incoming Truck	0.174	-0.094	-	-
	Bullock Cart	0.448	-0.214	-0.271	-0.053
Kalpi Road	Transit Truck	0.092	-0.0027	-	-
	Incoming Truck	0.079	-0.032	-	-
	Bullock Cart	0.403	-0.124	-0.224	-0.083
Hamirpur Road	Incoming Truck	0.251	-0.113	-	-
	Bullock Cart	0.264	-0.107	-0.125	-0.034

$$Z(t) = f'_2[Z(t-1), Z(t-2), \ldots\ldots Z(t-k), t] + f''_2[Y(t-1), Y(t-2), \ldots Y(t-m), t] \quad (3.10)$$

where f'_2 is due to the internal dependence and f''_2 is due to external dependence.

Due to the presence of seasonal cycles, the time series are nonstationary. It is necessary to transform them into stationary series before fitting the persistence models (Thomas and Fiering, 1962; Mckerchar and Delleur, 1972). The following transformation was used in this study;

$$X_2(t) = \frac{X_1(t) - \overline{X}_1(j)}{S(j)} \quad (3.11)$$

where $\overline{X}_1(j)$ and $S(j)$ are the mean and standard deviation of the logarithms of the trend free series $Z_1(t)$ in month j, j= 1, 2, 3, ..., 12. $\overline{X}_1(j)$ and $S(j)$ were estimated by maximum liklihood method. The transformed $X_2(t)$ series having zero mean and unit variance is stationary in mean and variance. The standardised series $X_2(t)$ were analysed for persistence.

Model Identification: Before a time series model may be fitted to the data, it is necessary to postulate or identify theoretical forms of the model. Correlogram, partial autocorrelation function (pacf) and power spectra are used for this purpose. For any given model, its correlogram, pacf and/or power spectra have specific characteristics. These

may be compared with the empirical values estimated from available data to identify a possible theoretical model that can be used for representing the time series. Persistence may be represented by Autoregressive (AR) and/or Moving Average (MA) models. Only AR model is described in this section. MA model is described later in Subsec. 3.4.3.

An AR process of order p (Box and Jenkins, 1970) may be represented by;

$$X_2(t) = \phi_1 X_2(t-1) + \phi_2 X_2(t-2) + \ldots \\ + \phi_p X_2(t-p) + a_t \qquad (3.12)$$

a_t is the random component and the other terms represent the persistence component, i.e., the present value of $X_2(t)$ depends upon the preceeding p values.

Let B = backward shift operator i.e., B $X_2(t) = X_2(t-1)$ and $(1-B)\ X_2(t) = X_2(t) - X_2(t-1)$. The AR operator of order p is defined by;

$$\phi(B) = 1 - \phi_1 B - \phi_2 B^2 - \ldots - \phi_p B^p \qquad (3.13)$$

So the AR model may be written as;

$$\phi\ (B)\ X_2(t) = a_t \qquad (3.14)$$

The model contains (p + 2) unknown parameters, ie., ϕ_1, ϕ_2, ..., ϕ_p, μ and σ_a^2 (variance of the random series a_t).

The autocorrelation function (acf) and pacf were computed for each of the standardised series $X_2(t)$. They are shown in Fig. 3.9 for incoming truck traffic on Lucknow road alongwith the 95 percent confidence limits. The decaying exponential behaviour of the estimated correlogram as well as significant partial autocorrelation at lag one identifies a first order AR model to represent the time series $X_2(t)$ i.e.,

$$X_2(t) = \phi_1 X_2(t-1) + a_t \tag{3.15}$$

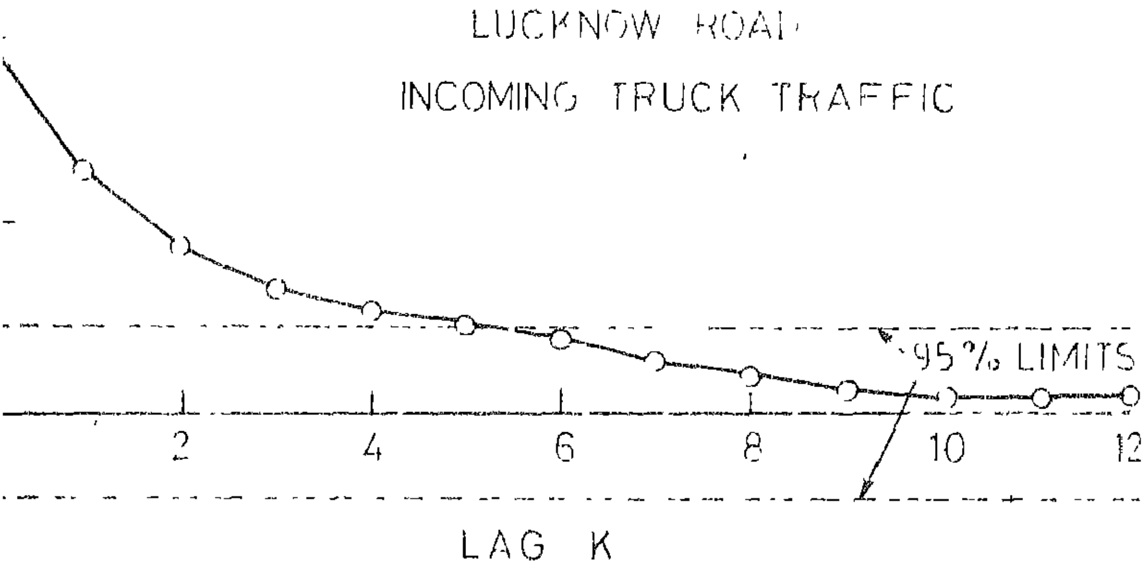
Estimation of Parameters: It is necessary to estimate the values of the parameters of the identified model from available data. For an AR process, ϕ coefficients can be initially estimated by Yule-Walker equations (Mckerchar and Delleur, 1972). Let r_1, r_2,... etc, be the acf for lags 1, 2 ......etc.

Then, for $\quad p = 1, \quad \phi_1 = r_1 \quad (3.16)$

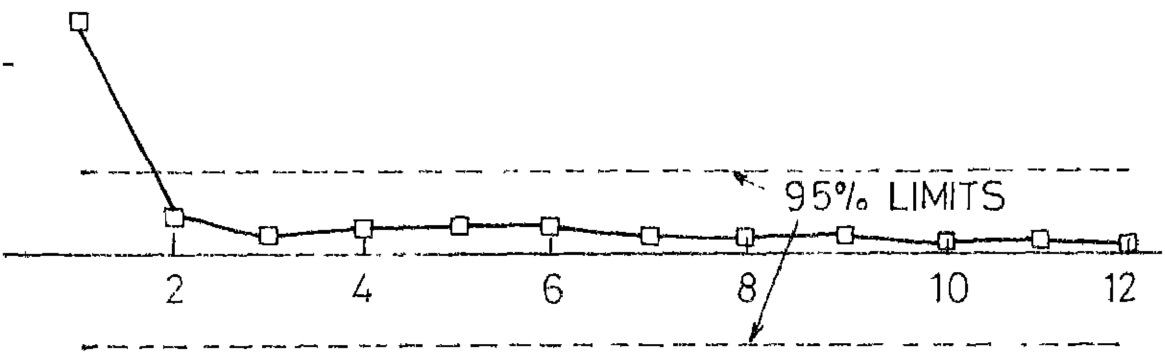
and for $\quad p = 2, \quad \phi_1 = \dfrac{r_1(1-r_2)}{1 - r_1^2} \quad$ and $\quad (3.17)$

$$\phi_2 = \frac{r_2 - r_1^2}{1 - r_1^2} \tag{3.18}$$

Initial estimate of the residual variance $\hat{\sigma}_a^2$ for the AR process may be obtained as follows (Box and Jenkins, 1970):



(a) ESTIMATED AUTOCORRELATIONS



(b) ESTIMATED PARTIAL AUTOCORRELATIONS

ESTIMATED AUTOCORRELATIONS OF TRANSFORMED SERIES

$$\sigma_a^2 = C_o (1 - \phi_1 r_1 - \phi_2 r_2 - \ldots\ldots - \phi_p r_p) \tag{3.19}$$

where $$C_o = \frac{1}{N} \sum_{i=1}^{N} (X_2(i) - \overline{X}_2)^2 \tag{3.20}$$

For the first order model identified for data series, parameter estimates are obtained as follows:

$$\phi_1 = r_1$$

$$\sigma_a^2 = C_o (1 - \phi_1 r_1) = C_o - \phi_1 C_1 \tag{3.22}$$

Box and Jenkins (1965, 1970) suggest a nonlinear iterative algorithm that leads to the maximum liklihood estimation of the model parameters by minimising the variance of the residuals, viz., the residual sum of squares Σa_t^2. This procedure was adopted, with the estimates from Yule-Walker relations as initial values, and the parameters were determined. The initial values and successive estimates for bullock cart traffic on Delhi road are given in Table 3.3. Convergence is reasonably fast; and the initial values estimated are not far from the final values in this case. The final estimates for all the cases are listed in Table 3.4 . Residual variances lie between 0.498 and 0.816 indicating that between 18.4 to 50.2 percent of variance is explained by persistence.

TABLE 3.3 ITERATIVE ESTIMATION OF PARAMETERS

(Delhi Road - Bullock cart traffic)

Iteration	ϕ_1	Sum of Squares
Starting value	0.2200	29.39
1	0.1952	24.01
2	0.1913	23.98
3	0.1887	23.97
Final value	0.1887	23.97

Diagnostic Checking: It refers to the checking and validation of the fitted model and if it is not adequate, the determination of inadequacies to indicate a better model. The techniques normally used are (i) overfitting and (ii) examining the residuals.

Overfitting means fitting a more elaborate model than identified and comparing the fits in terms of the residual sum of squares of the more elaborate and less elaborate models thus giving an indication as to whether the model identified may be improved upon or not. Examining the residuals gives a more general check on the adequacy of the model. If the model is adequate and estimated parameters

TABLE 3.4 FINAL ESTIMATES OF PARAMETERS FOR FIRST ORDER AR MODEL

Road	Category of Traffic	σ_a^2	ϕ_1	Q (23 DOF)
Delhi Road	Transit	0.743	0.3713	25.82
	Incoming Truck	0.624	0.6404	30.73
	Bullock Cart	0.778	0.1887	18.55
Allahabad Road	Transit	0.785	0.2876	24.98
	Incoming Truck	0.602	0.4732	8.86
	Bullock Cart	0.516	0.3179	25.27
Lucknow Road	Transit	0.730	0.1917	35.21*
	Incoming Truck	0.592	0.4276	30.30
	Bullock Cart	0.758	0.3938	21.87
Kalpi Road	Transit	0.728	0.3854	11.57
	Incoming Truck	0.498	0.5579	22.85
	Bullock Cart	0.617	0.4445	20.73
Hamirpur Road	Incoming Truck	0.528	0.4854	33.38*
	Bullock Cart	0.816	0.5728	20.26

χ^2_{23} (10 %) = 32.0 χ^2_{23} (5 %) = 35.20

* Significant at 10 % level but not at 5 % level

are close to the true values, the residuals a_t should be a purely random series. If a_t series are not purely random but show some serial dependence, the tentatively identified model is inadequate and it is to be modified. The nature of the dependence gives some indication of the modification that need be made. In the present study, diagnostic checking was done by examining the residuals. The two checks that were employed are (i) autocorrelation of the residuals and (ii) cumulative periodogram of the residuals.

The residual autocorrelation, r_i, should be independent and normally distributed about zero with variance 1/N. For lower lags and for short series, the estimates of r_1 and r_2 are biased and it is not proper to use the standard error as a basis for determining whether the correlogram represents a random series. Instead a statistic $Q = N \sum_{i=1}^{k} r_i^2$ can be calculated for the first k values of autocorrelations of the residuals, to indicate the randomness or otherwise of the residual series(Durbin, 1969; Box and Pierce, 1970). For the truly random series, Q is distributed as χ^2 with (N - p - q) degrees of freedom where p and q are the orders of AR and MA components respectively.

For seasonal time series, acf of residuals may not be a sensitive indicator of the departures from randomness, because periodic effects will typically dilute themselves among several autocorrelations. The periodogram can be used for detecting the periodic pattern in the background of white noise (Barlett, 1955) . The power spectra $p\ (f)$ for white noise has a constant value $2\ \sigma_a^2$ over the frequency domain 0-0.5 cycles. The cumulative spectrum for white noise $P\ (f) = \int_0^f p\ (g)\ d\ g$ plotted against f is a straight line running from (0 , 0) to ($0.5, \sigma_a^2$) ie., $P\ (f) / \sigma_a^2$ is a straight line running from (0 , 0) to (0.5 , 1),

$$C\ (f_j) = \frac{\sum_{i=1}^{j} I\ (f_i)}{n\ (sd)^2} \qquad (3.23)$$

provides an estimate of $P(f_j) / \sigma_a^2$ where $(sd)^2$ is an estimate of σ_a^2 and $I\ (f_j)$ an estimate of $P\ (f_j)$. The cumulative periodogram is the sample estimate of the integrated spectrum. On the basis of Kolmogorov- Smirnov test (Hald, 1952), the confidence limits of the cumulative periodogram are given by $\pm\ K_e / \sqrt{q}$ where K_e is the coefficient for calculating probability limits and $q = (N - 2) / 2$ for N even and $(N - 1) / 2$ for N odd.

The autocorrelations for the residual a_t series were computed. They were not significant at 95 percent confidence levels, thereby indicating serial independence. Values of statistic Q are shown in Table 3.4 for all the series. These values were generally not significant compared with χ^2 values at 10 percent level of significance. Only in two cases, Q values were significant at 10 percent level but not at 5 percent level. The cumulative periodogram for the residual a_t series is shown in Fig. 3.10 alongwith 5 and 25 percent probability limits. They also confirm that the residual series do not have serial dependence. The fit of the models is hence good and they are accepted.

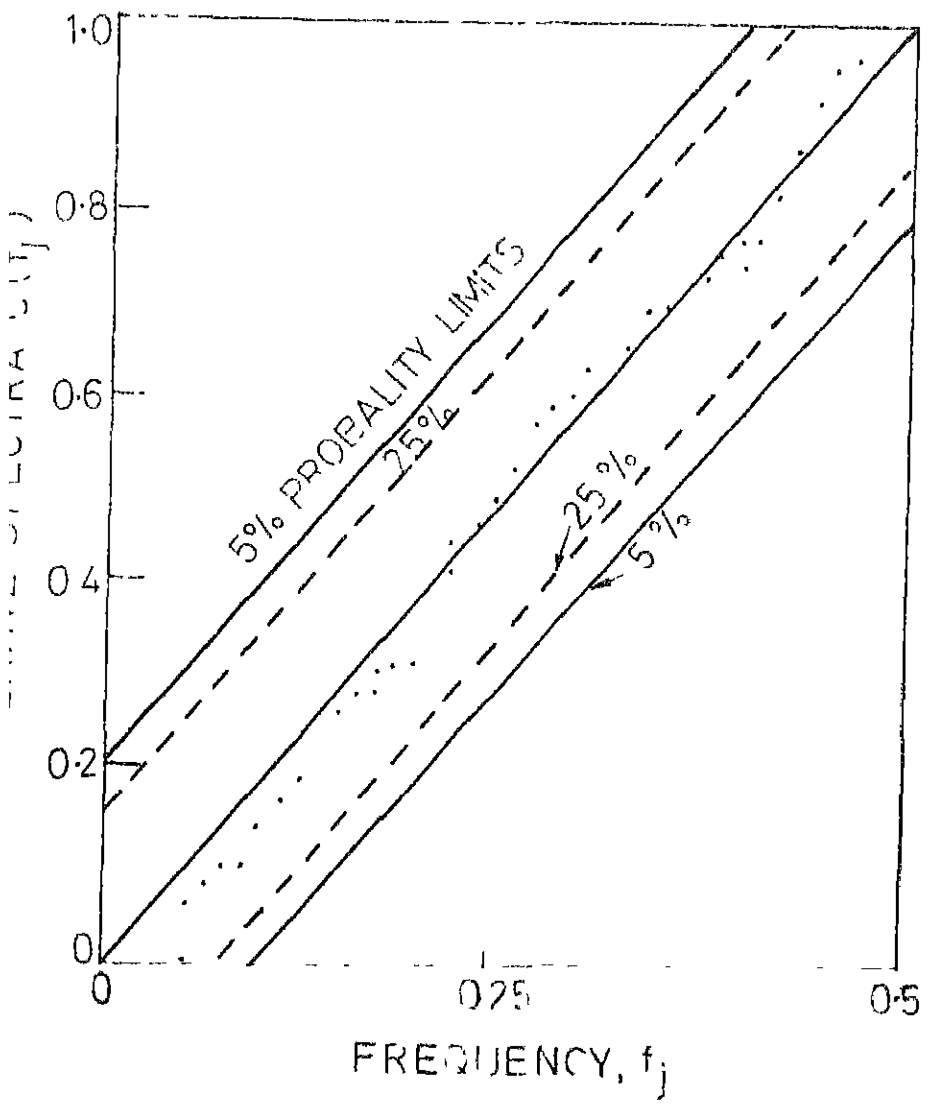
3.4.3 Seasonal ARIMA Model

The autoregressive model has already been presented in Subsec. 3.4.2. The deviation of a process from mean, say, $w_t = (Z(t) - \bar{Z})$, may be represented by a moving average process (Box and Jenkins, 1970) of order q, viz.,

$$w_t = Z(t) - \bar{Z} = a_t - \theta_1 a_{t-1} - \theta_2 a_{t-2} - \ldots \ldots - \theta_q a_{t-q} \qquad (3.24)$$

where θ_1, θ_2, ..., θ_q are a finite set of parameters and a_t is a random series with zero mean and variance σ_a^2. The process can also be represented in

ALLAHABAD ROAD
INCOMING TRUCK TRAFFIC



3·10 CUMULATIVE PERIODOGRAM OF RESIDUAL AUTOCORRELATIONS

terms of backward shift operator, B by

$$w_t = Z(t) - \bar{Z} = (1 - \theta_1 B - \theta_2 B^2 - .... - \theta_q B^q) a_t$$

$$= \theta (B) a_t \qquad (3.25)$$

where θ (B) is a polynominal in B of order q .

A more general model is the Autoregressive Moving Average (ARMA) model of order (p , q) with both AR and MA components, viz.,

$$w_t = Z(t) - \bar{Z} = \phi_1 w_{t-1} + \phi_2 w_{t-2} + ... + \phi_p w_{t-p}$$

$$+ a_t - \theta_1 a_{t-1} - \theta_2 a_{t-2} - .... - \theta_q a_{t-q} \qquad (3.26)$$

or $$\phi (B) (Z (t) - \bar{Z}) = \theta (B) a_t \qquad (3.27)$$

The series of traffic volume generally exhibit a trend along time, representing growth, and seasonal variations. When trend is exhibited, it may be possible to represent nonstationarity in terms of differences rather than in terms of the original time series. ARIMA process assumes that the dth degree of differencing of the series can be represented by an ARMA process of order (p , q), namely $\nabla^d = (1 - B)^d$ and

$$\phi_p(B) \nabla^d (Z(t) - \bar{Z}) = \theta_q (B) a_t \qquad (3.28)$$

is referred to as an ARIMA process (Mckercher and Delleur, 1972)

of order (p , d , q).

When seasonal effects alone are present, they may be eliminated by using seasonal differencing. Let s = period of differencing (s = 12 for monthly series data with an annual cycle); $\nabla_s^D = (1 - B^s)^D$; where D = degree of seasonal differencing. The time series may thus be represented by ARIMA model (Bacon, 1965) of order $(P , D , Q)_s$

$$\Phi_P(B^s) \quad \nabla_s^D \; (Z(t) - \bar{Z}) = \Theta_Q(B^s) \; e_t \qquad (3.29)$$

where

$$\Phi_P(B^s) = (1 - \Phi_1 B^s - \Phi_2 B^{2s} - \ldots - \Phi_P B^{Ps}) ,$$

a seasonal autoregressive operator of order P, and

$$\Theta_Q (B^s) = (1 - \Theta_1 B^s - \Theta_2 B^{2s} - \ldots - \Theta_Q B^{Qs}),$$

a seasonal moving average operator of order Q.

When both trend and seasonal effects are present, the multiplicative seasonal ARIMA model of order $(p , d , q) \times (P , D , Q)_s$ can be represented by

$$\Phi_P (B^s) \; \phi_p(B) \quad \nabla_s^D \quad \nabla^d \; (Z(t) - \bar{Z}) = \Theta_Q(B^s) \; \theta_q(B) \; a_t \qquad (3.30)$$

ARIMA models have the advantage of representing complex time series with only a relatively small number of parameters.

Model Identification: Logarithms (natural) of monthly traffic flows tend to be normally distributed and were used for further analysis. Autocorrelations were estimated for the following cases: (i) undifferenced series, i.e., original log transformed series; (ii) first differencing of the series w.r.t months only, i.e., $d = 1$; (iii) first seasonal differencing of the series, i.e., $D = 1$ with $s = 12$ to take into account the annual cycle; and (iv) first seasonal and serial differencing of the series, i.e., $d = 1$, $D = 1$.

The estimated autocorrelations of differenced series for transit traffic on Lucknow road are shown in Table 3.5. It is seen that for undifferenced series as well as for simple differencing ($d = 1$) and for serial and seasonal differencing ($d = 1$, $D = 1$), serial correlations are not negligible for a number of lags and they fail to die out at higher lags. By contrast only seasonal differencing ($D = 1$) alone gives an improved result. The autocorrelations of this series ($D = 1$) are statistically significant at $\pm \frac{2}{\sqrt{N}}$ confidence limits only for lag 12. The insignificant values between lags 2-11 and large values at lags 1, 12, 13, 14, 24, 25 and 26 indicate a multiplicative seasonal ARIMA process of order $(1, 0, 0) \times (0, 1, 1)_{12}$ i.e., $p = Q = 1$, $P = q = 0$, $d = 0$, and $D=1$. It is of the form

TABLE 3.5 ESTIMATED AUTOCORRELATION FUNCTIONS FOR DIFFERENCED SERIES (TRANSIT TRAFFIC - LUCKNOW ROAD)

d	D	n	Lags	Estimated Autocorrelation Functions						Confidence Limits
				1	2	3	4	5	6	$\pm \frac{2}{\sqrt{n}}$
0	0	34	1-6	.25*	.12	.13	-.10	-.12	.02	± 0.218
			7-12	-.11	-.17	.13	.16	.15	.38*	
			13-18	.27*	.05	.05	-.10	-.23*	-.02	
			19-24	-.11	-.15	-.09	.04	.03	.25*	
			25-30	.04	-.10	.06	-.12	-.15	-.15	
			31-36	-.13	-.19	-.01	.01	-.07	.16	
1	0	83	1-6	-.40*	-.09	.12	-.12	-.10	.20	± 0.218
			7-12	-.06	-.23*	.18	.01	-.16	.24*	
			13-18	.06	-.13	.10	-.02	-.22*	.23*	
			19-24	-.07	-.05	-.06	.08	-.17	.32*	
			25-30	-.06	-.17	.23*	-.11	-.02	-.02	
			31-36	.05	-.15	.10	.05	-.20	.20	
0	1	72	1-6	.12	.02	-.00	.05	.06	.03	± 0.235
			7-12	.01	-.03	.10	.07	-.03	-.35*	
			13-18	.11	.15	-.01	.02	-.11	.12	
			19-24	.01	.01	-.12	-.06	.03	.13	
			25-30	-.17	-.19	.08	.05	.07	-.08	
			31-36	.01	-.03	.06	.05	-.14	.03	
1	1	71	1-6	-.44*	-.04	-.08	-.00	.13	-.07	± 0.235
			7-12	.03	-.19	.28	-.06	.13	-.45*	
			13-18	.24*	.11	-.10	.07	-.20	.21	
			19-24	-.08	.14	-.24*	.05	.05	.13	
			25-30	-.12	-.16	.17	-.02	.10	-.15	
			31-36	.08	-.07	.06	.10	-.21	.08	

* Significant at 95 % confidence level

$$(1 - \phi_1 B) \nabla_{12} w_t = (1 - \theta_1 B^{12}) a_t$$

where

$$w_t = X(t) - \bar{X} (t) \text{ with } X(t) = \mathrm{Log}_e Z(t) \qquad (3.31)$$

Various other series also gave similar results.

Estimation for (1, 0 , 0) x (0, 1 , 1)$_{12}$ Model:- The model parameters were estimated by a nonlinear iterative algorithm (Box and Jenkins, 1970) that minimises the sum of squared errors. Values of the parameters were estimated as follows:

Sum of square functions S(ϕ , θ) were computed for each time series over a range of ϕ_1 and θ_1 , values. A plot for transit traffic on Lucknow road is shown in Fig. 3.11. A minimum is indicated at $\phi_1 = 0.3$, $\theta_1 = 0.7$. These values can be used as starting values for estimation of parameters by the iterative procedure. Table 3.6 shows that the iterative process converges quickly. The final estimates are close to the initial estimates.

The final estimates of the parameters for other series were also obtained by the same procedure and are given in Table 3.7.

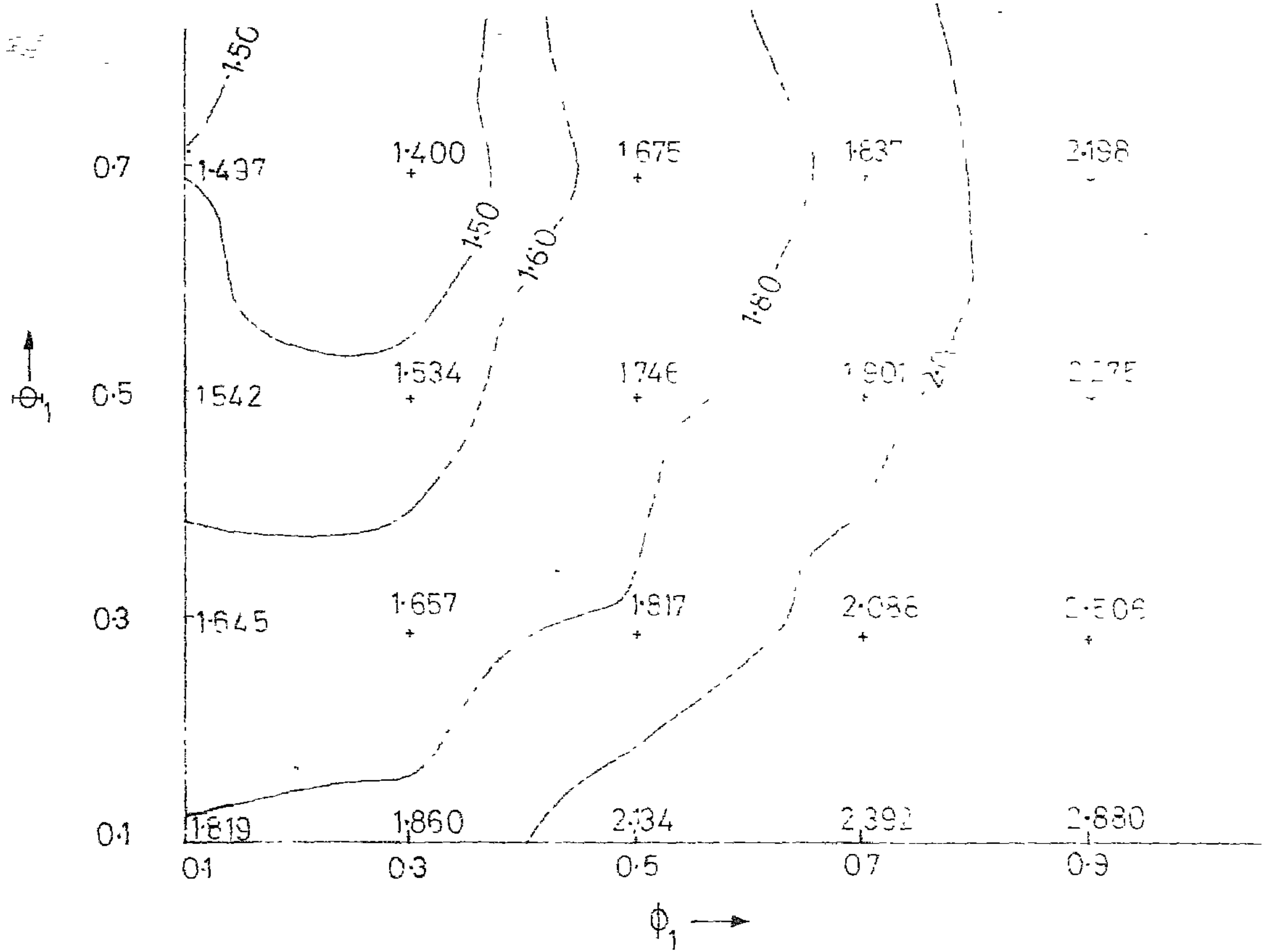


FIG. 3·11 SUM OF SQUARES SURFACE $S(\phi_1, \Theta_1)$

TABLE 3.6 ITERATIVE ESTIMATION OF PARAMETERS FOR ARIMA MODEL (TRANSIT TRUCK TRAFFIC - LUCKNOW ROAD)

Iteration	Sum of Squares	ϕ_1	Θ_1
Starting Value	1.40017	0.3000	0.7000
1	1.35495	0.2291	0.7774
2	1.32785	0.2325	0.8363
3	1.31971	0.2304	0.8652
4	1.31868	0.2247	0.8747
5	1.31861	0.2215	0.8778
Final Value	1.31861	0.2215	0.8778

Diagnostic Checking: The adequacy of the fit of the model was checked by examining the residuals from the fitted model. Autocorrelations for any series were not significant at 95 percent confidence level. The Q statistic is given for all the series in Table 3.7. Compared with the critical value of χ^2 at 10 percent significance level and 22 Degrees of freedom (= 30.8) , only one Q value is marginally significant though it is also not significant at 5 percent significance level. The residual series may thus be considered to be pure random. Hence the multiplicative seasonal ARIMA model of order (1, 0, 0) x (0, 1, 1)$_{12}$

TABLE 3.7 FINAL ESTIMATES OF PARAMETERS FOR MULTIPLICATIVE SEASONAL ARIMA MODEL (1, 0, 0) x (0, 1, 1)$_{12}$

Road	Category of Traffic	σ_a^2	ϕ_1	θ_1	Q (22 DOF)
Delhi Road	Transit Truck	0.886	0.4163	0.7533	21.95
	Incoming Truck	0.387	0.6822	0.9117	25.72
	Bullock Cart	0.670	0.1524	0.7664	10.87
Allahabad Road	Transit Truck	0.730	0.3007	0.7914	14.11
	Incoming Truck	0.546	0.4591	0.8326	16.28
	Bullock Cart	0.435	0.3488	0.6255	33.74*
Lucknow Road	Transit Truck	0.508	0.2215	0.8778	27.83
	Incoming Truck	0.406	0.3837	0.7060	28.54
	Bullock Cart	0.178	0.4016	0.5884	18.06
Kalpi Road	Transit Truck	0.728	0.4349	0.7969	15.82
	Incoming Truck	0.917	0.5110	0.9082	27.54
	Bullock Cart	0.453	0.4253	0.8193	23.92
Hamirpur Road	Incoming Truck	0.803	0.4691	0.8375	21.35
	Bullock Cart	0.956	0.5134	0.7586	24.66

χ^2_{22} (10 %) = 30.8 χ^2_{22} (5 %) = 33.92

* Significant at 10 % level but not at 5 % level

appears to fit the log transformed traffic flow series well.

3.4.4 Traffic Flow Forecasting

Stochastic models for monthly traffic volumes can be used to forecast future values of the time series. The approach to forecasting is described and the traffic forecasts using seasonal ARIMA model on Delhi road, upto April 1976 are also presented.

The objective is to make a forecast of $Z(t+L)$ (or its transform w_{t+L}), at time t for $L \geq 1$. L is the lead time and corresponding forecast is known as L - step ahead forecast denoted respectively by $\hat{Z}_t(L)$ or $\hat{w}_t(L)$. Box and Jenkins (1970) show that the minimum mean square error forecast $\hat{w}_t(L)$ is the conditional expectation of w_{t+L} at time t ,i.e.,

$$\hat{w}_t(L) = E_t[w_{t+L}] \tag{3.32}$$

Further, the error of estimate of the forecast is

$$\text{Error}_t(L) = w_{t+L} - \hat{w}_t(L) \tag{3.33}$$

The above error function can also be written as (Barlett, 1955):

$$\text{Error}_t(L) = a_{t+L} + \psi_1 a_{t+L-1} + \cdots\cdots + \psi_{L-1} a_{t+1} \tag{3.34}$$

With $L = 1$, $\text{Error}_t(1) = a_{t+1} = w_{t+1} - \hat{w}_t(1)$, that is, a_t

series are one step ahead forecast errors.

For the seasonal ARIMA model with p = 1 , d = q = P = 0, D = Q = 1,

$$(1 - \phi_1 B) (1 - B^{12}) (\psi_0 + \psi_1 B + \psi_2 B^2 + \ldots.)$$
$$= 1 - \Theta_1 B^{12} \qquad (3.35)$$

or

$$(1-\phi_1 B - B^{12} + \phi_1 B^{13}) (\psi_0 + \psi_1 B + \psi_2 B^2 + \ldots..)$$
$$= 1 - \Theta_1 B^{12} \qquad (3.36)$$

Thus equating the coefficients of B

$$\psi_0 = 1$$
$$\psi_1 = \phi_1$$
$$\psi_2 = \phi_1 \psi_1$$
$$\cdot$$
$$\cdot$$
$$\cdot$$
$$\psi_{11} = \phi_1 \psi_{10} \qquad (3.37)$$
$$\psi_{12} = \phi_1 \psi_{11} + 1 - \Theta_1$$
$$\psi_{13} = \phi_1 \psi_{12} + \psi_1 - \phi_1$$
$$= \phi_1 \psi_{12}$$
$$\psi_j = \phi_1 \psi_{j-1} + \psi_{j-12} - \phi_1 \psi_{j-13} \quad \text{for } j \geq 13$$

The variance of the forecast error can be written as:

$$V[L] = \mathrm{Var}[\mathrm{Error}_t(L)] = (1 + \psi_1^2 + \psi_2^2 + \cdots + \psi_{L-1}^2)\ \sigma_a^2 \qquad (3.38)$$

The general ARIMA model can be written in three alternative forms:

(i) as a difference equation;

(ii) as an infinite sum of the current and previous values of shocks a_t; and

(iii) as an infinite sum of previous observations plus the current value of a_t.

Conditional expectation of any of these forms gives a forecasting function . In this study, forecasts were obtained using the difference equation form.

The developed model is of the form ;

$$(1-\phi_1 B)(w_{t+L} - w_{t+L-12}) = (1 - \theta_1 B^{12})\ a_t \qquad (3.39)$$

Taking expectations,

$$[w_{t+L}] = \phi_1 [w_{t+L-1}] + [w_{t+L-12}] - \phi_1 [w_{t+L-13}] + [a_{t+L}] - \theta_1 [a_{t+L-12}] \qquad (3.40)$$

Thus, for

$$L = 1 ;\ \hat{w}_t(1) = \phi_1 w_t + w_{t-11} - \phi_1 w_{t-12} - \theta_1 a_{t-11}$$

$$L = 2,3,\ldots 12: \hat{w}_t(L) = \phi_1 \hat{w}_t(L-1) + w_{t+L-12} - \phi_1 w_{t+L-13} - \theta_1 a_{t+L-12}$$

$$L = 13: \quad \hat{w}_t(13) = \phi_1 \hat{w}_t(12) + \hat{w}_t(1) - \phi_1 w_t$$

$$L = 14, 15, \ldots : \hat{w}_t(L) = \phi_1 \hat{w}_t(L-1) + \hat{w}_t(L-12) - \phi_1 \hat{w}_t(L-13) \qquad (3.41)$$

The standard error of $\hat{w}_t(L)$ is

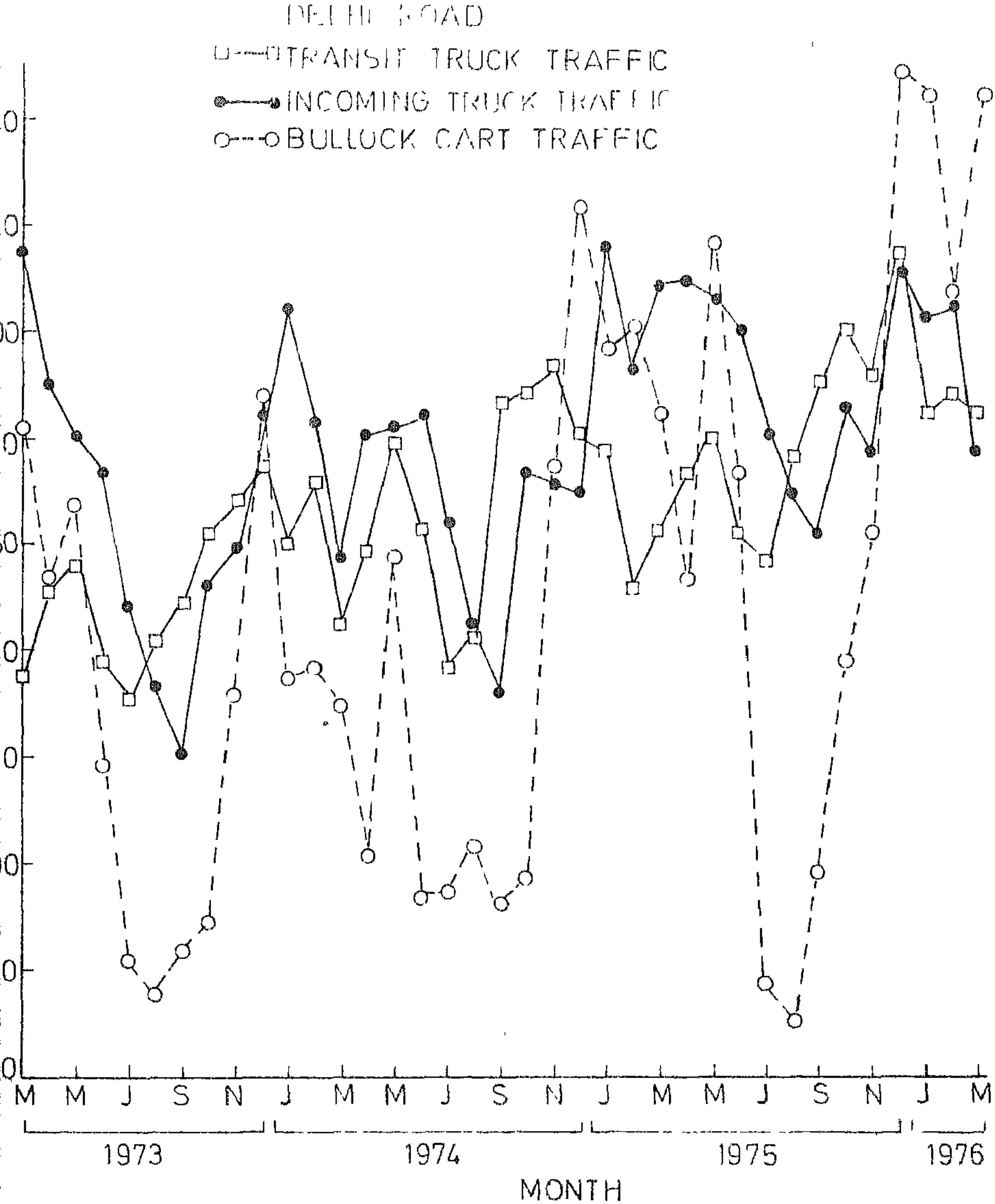
$$S_{\hat{W}}(L) = (1 + \psi_1^2 + \psi_2^2 + \ldots\ldots + \psi_{L-1}^2)^{1/2} \sigma_a \qquad (3.42)$$

The function $\hat{w}_t(L)$ gives the minimum mean square error forecasts of $X_t(L) - \bar{X}_t$ where $X_t(L)$ is the logarcthm of $Z(t+L)$. $\hat{X}_t(L)$ is a conditional mean which is expected to be normally distributed about the actual flow logarithms with standard deviation $S_{\hat{W}}(L)$. Using the method of moments (Mekercher and Dellur, 1972)

$$\hat{Z}_t(L) = \exp\left[\hat{X}_t(L) + S_{\hat{W}}^2(L)/2\right] \qquad (3.43)$$

$$S_t(L) = \hat{Z}_t(L)\left[\exp\left(S_{\hat{W}}^2(L)\right) - 1\right]^{1/2} \qquad (3.44)$$

In this study, traffic forecasts of three different categories of goods carriers were made only for Delhi road (Fig. 3.12) using seasonal ARIMA model. This model involves much less number of parameters compared to the general time series model. The forecasts were made for 36 months ahead of the last available value (March 1973) of the series. The



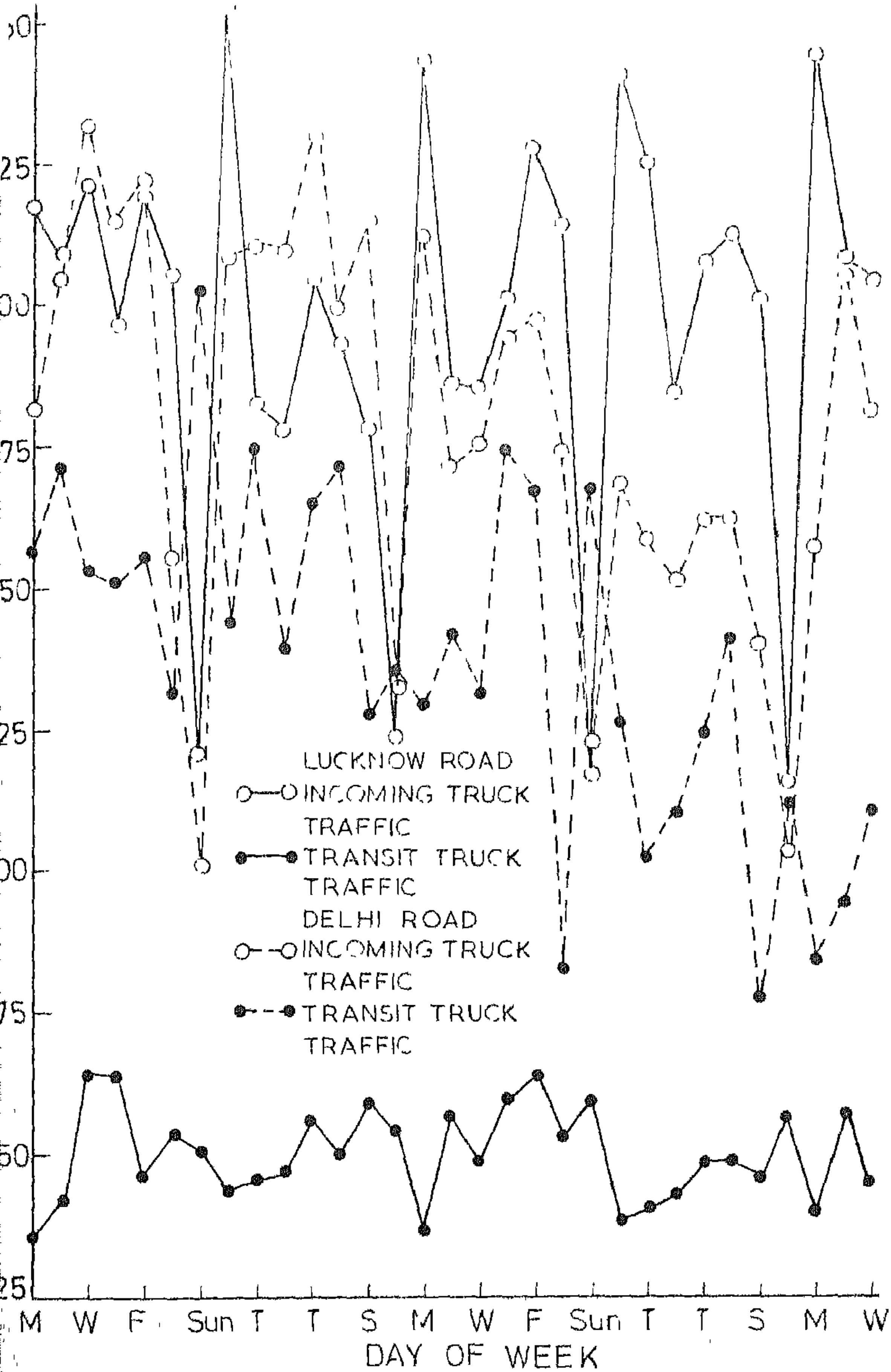
3·12 TRAFFIC FORECASTS BY SEASONAL ARIMA MODEL

actual traffic volume data beyond March 1973 could not be obtained due to changed pattern of octroi collection. Thus the adequacy of forecasts could not be checked though their general characteristics are similar to historical data.

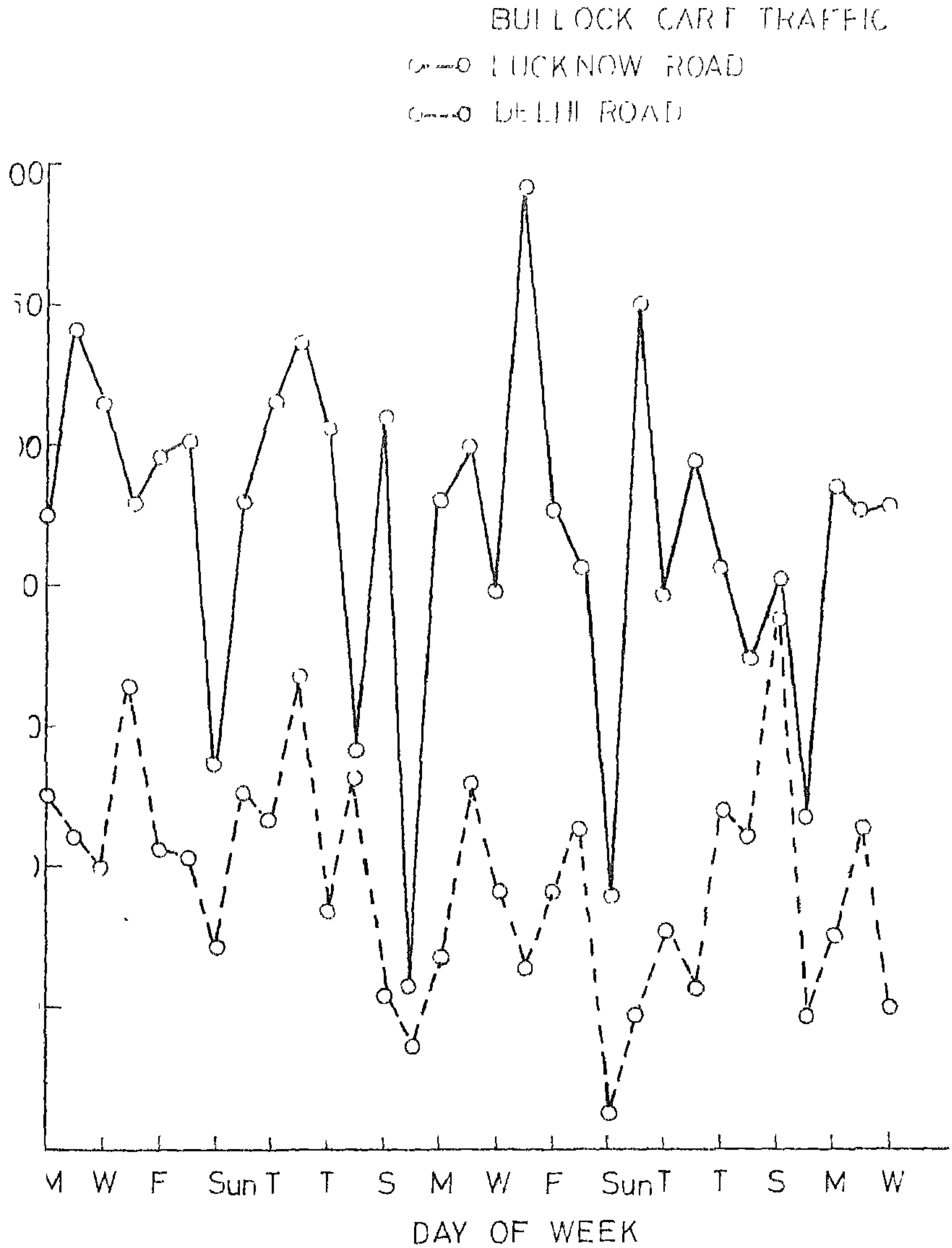
3.5 Analysis of Daily Traffic Flows Within a Month

Time series models of monthly traffic flows consider the average daily traffic flows of various months. It is also desireable to study the daily traffic volumes within a month so as to estimate the daily traffic demand. Stochastic models have been developed in this study for a few representative months. Analysis was carried out for the daily traffic data on two roads, namely, Delhi road and Lucknow road , during the peak traffic months of December, 1972, January, 1973 and February, 1973.

Figs. 3.13 and 3.14 show the daily volumes of three categories of goods carriers during January, 1973. These figures indicate that all categories of goods traffic fluctuate throughout the month. Fig. 3.13 shows that incoming truck traffic volume was minimum on Sundays in both roads. The peak occurs generally on Monday in Lucknow road and sometime mid-week in Delhi road . For the transit truck traffic (Fig. 3.13) on Lucknow road, the daily



313 DAILY TRAFFIC VARIATIONS WITHIN THE MONTH (DEC, 1972)



4 DAILY TRAFFIC VARIATIONS WITHIN THE MONTH (DEC. 1972)

variations are smaller with the minimum volume occuring generally on Mondays. On the Delhi road, transit truck traffic also fluctuates throughout the week with no marked peak or minimum on a particular day. Bullock cart traffic volume shown in Fig. 3.14 indicates minimum traffic flow on Sundays similar to incoming truck traffic. The above results indicate that there are marked daily variations within the week for each category of goods traffic.

Daily traffic volume data were normalised by log transformation to X (t) series. Autocorrelations and partial autocorrelations were estimated for the following three cases:

(i) undifferenced log transformed series X(t) ;

(ii) serial differencing of the X(t) series w.r.t days only i.e., d = 1; and

(iii) weekly differencing of the series ie., D = 1 and s = 7 days .

It was observed that for case (iii), estimated autocorrelations are smaller than those of cases (i) and (ii). The log transformed series thus need to be weekly differenced before identifying the nature of the model.

The estimated autocorrelations of all the series are shown in Table 3.8. For one series, acf and pacf are also shown in Fig. 3.15. The decaying exponential behaviour

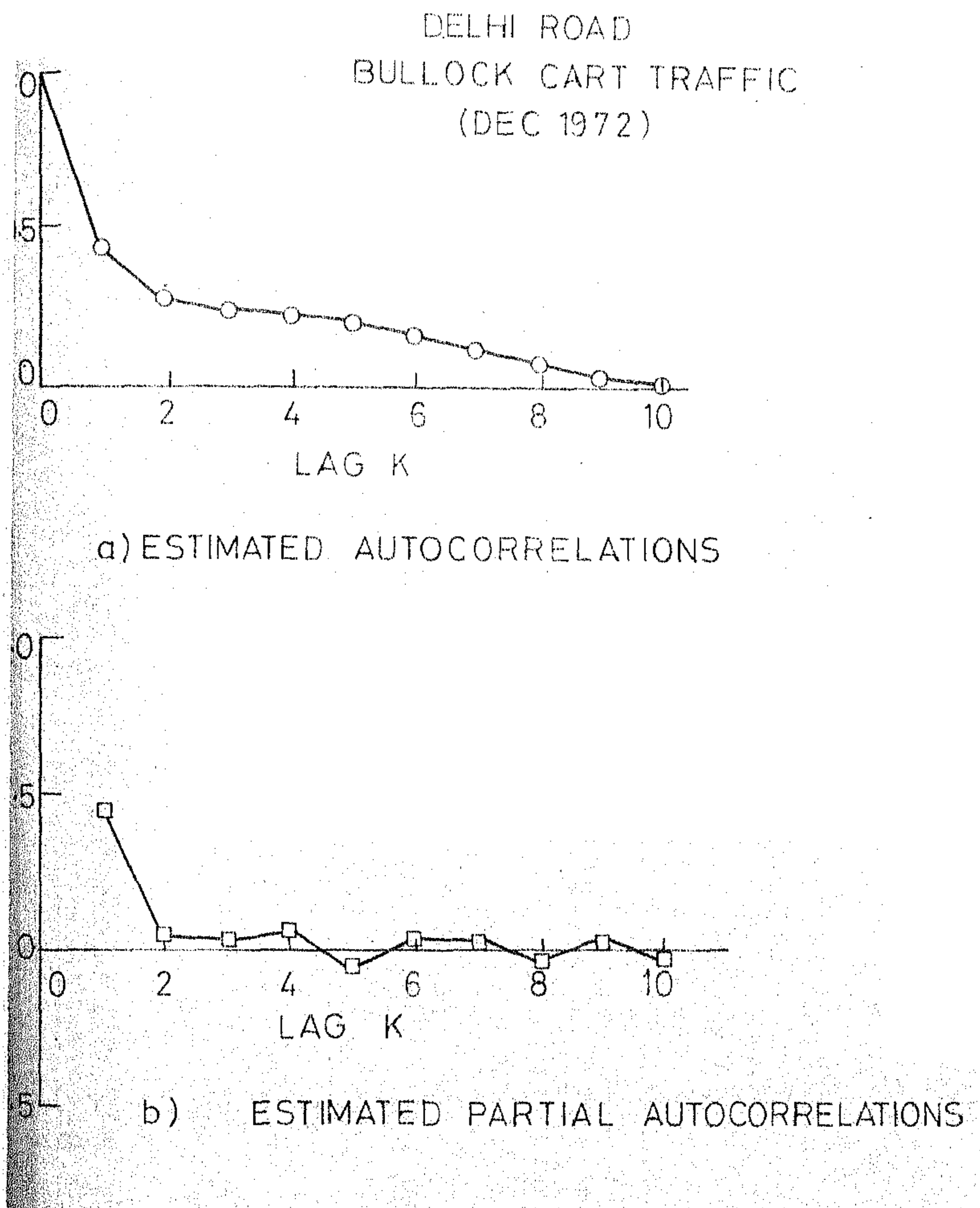
TABLE 3.8 ESTIMATED AUTOCORRELATION FUNCTIONS FOR WEEKLY DIFFERENCED DATA SERIES

Road	Category of Traffic	Month	Lags	Estimated Autocorrelation Functions				
				1	2	3	4	5
Delhi Road	Transit Truck	12/72	1-5	.28	.16	.19	.15	.13
			6-10	.11	.09	.05	.06	.01
		1/73	1-5	.31	.25	.19	.13	.17
			6-10	.10	.10	.07	-.04	.00
		2/73	1-5	.27	.12	.16	.11	.07
			6-10	-.03	.04	.01	-.00	-.02
	Incoming Truck	12/72	1-5	.35	.25	.21	.18	.13
			6-10	.15	.10	-.03	-.05	.02
		1/73	1-5	.32	.29	.22	.24	.20
			6-10	.14	.13	.06	.07	.05
		2/73	1-5	.34	.25	.21	.17	.19
			6-10	.13	.11	.09	-.06	.02
	Bullock Cart	12/72	1-5	.45	.29	.25	.22	.20
			6-10	.16	.11	.07	.03	.01
		1/73	1-5	.42	.30	.27	.21	.19
			6-10	.13	.10	.10	.08	-.04
		2/73	1-5	.44	.34	.34	.24	.20
			6-10	.15	.11	.05	-.07	.02

Contd.....

TABLE 3.8 CONTD....

Road	Category of Traffic	Month	Lags	Estimated Autocorrelation Functions				
				1	2	3	4	5
Lucknow Road	Transit Truck	12/72	1-5	.12	.11	.09	.10	.07
			6-10	.02	-.04	.03	.01	.00
		1/73	1-5	.10	.07	.04	.02	.01
			6-10	-.01	.00	.02	.01	-.03
		2/73	1-5	.11	.09	.10	.06	.03
			6-10	.01	-.03	.02	-.01	.00
	Incoming Truck	12/72	1-5	.39	.31	.21	.26	.20
			6-10	.14	.09	.04	.01	-.03
		1/73	1-5	.40	.32	.25	.21	.16
			6-10	.10	.05	.04	.05	.02
		2/73	1-5	.36	.29	.23	.20	.21
			6-10	.16	.13	.09	-.02	.04
	Bullock Cart	12/72	1-5	.45	.25	.25	.18	.19
			6-10	.12	.11	.09	-.01	.00
		1/73	1-5	.48	.33	.25	.19	.13
			6-10	.18	.19	.11	.06	.01
		2/73	1-5	.48	.30	.24	.22	.12
			6-10	.16	.14	.08	.07	-.02



3·15 ESTIMATED AUTOCORRELATIONS AND PARTIAL AUTOCORRELATIONS OF WEEKLY DIFFERENCED SERIES

of acf and substantial pacf at lag one suggest that the first order AR model of the form $X(t) = \phi_1 X(t-1) + a_t$ may be suitable. Preliminary estimates of ϕ_1 and σ_a^2 were made using Yule-Walker relationship (Eqs. 3.16 and 3.19). Final estimates were obtained by minimizing the residual sum of squares (Subsec. 3.4.2) and are shown in Table 3.9. The final estimates are close to the preliminary estimates. The autocorrelations of the residual a_t series were calculated and very few values are significant compared with the confidence limits of $\pm \frac{2}{\sqrt{N}}$. Values of Q statistic for various series are also shown in Table 3.9. None of them is significant at 5 percent level and only one is significant at 10 percent level.

These results indicate that log transformed daily traffic series, after weekly differencing, can be adequately expressed by an AR process of first order. For peak months ϕ_1 values are very close to each other (Table 3.9) for the same traffic volume series and hence an average value of ϕ_1 may be used in analysis.

TABLE 3.9 FINAL ESTIMATES OF PARAMETERS FOR THE FIRST ORDER AR MODEL

Road	Category of Traffic	Month	ϕ_1	σ_a^2	Q (10 DOF)
Delhi Road	Transit Truck	12/72	0.29	0.774	10.92
		1/73	0.28	0.714	11.85
		2/73	0.29	0.627	3.27
	Incoming Truck	12/72	0.32	0.585	5.98
		1/73	0.36	0.392	9.86
		2/73	0.31	0.547	8.45
	Bullock Cart	12/72	0.42	0.736	13.92
		1/73	0.39	0.952	3.45
		2/73	0.46	1.293	17.82*
Lucknow Road	Transit Truck	12/72	0.11	0.607	5.95
		1/73	0.09	0.742	11.67
		2/73	0.12	0.738	12.54
	Incoming Truck	12/72	0.37	0.506	12.02
		1/73	0.41	0.607	10.05
		2/73	0.38	0.325	9.34
	Bullock Cart	12/72	0.43	0.985	11.35
		1/73	0.46	0.886	16.79
		2/73	0.47	0.891	5.44

χ^2_{10} (10 %) = 15.99 χ^2_{10} (5 %) = 18.31

* Significant at 10 % level but not at 5 % level

3.6 Analysis of Traffic Flows Within the Day

3.6.1 Hourly Variations

Traffic volume was recorded at 5 minute intervals for 10 days for all categories of incoming and outgoing vehicles on G.T. Road (Sec. 2.2). It was found that 5 minute volume data fluctuate randomly within an hour and follow a Poisson distribution for all categories of vehicles. Because of daily fluctuations within the week, the average daily traffic (ADT) for each category was calculated from data for one week only. The mean hourly volumes of different categories in terms of ADT were computed from field data for a week. They indicate the hourly variations within the day. They are shown for both incoming and outgoing directions for different categories of vehicles in Figs. 3.16 to3.18.

Analysis of daily data for car traffic in a week indicated that generally minimum volume occurs on Sunday for both incoming and outgoing directions and maximum occurs on Monday for incoming and on Saturday for outgoing directions. For cars (Fig. 3.16), outgoing traffic is maximum during 1000 to 1100 hours, whereas for incoming traffic, there are two peaks, one in the morning (1000 to 1100 hours) and the other in the evening

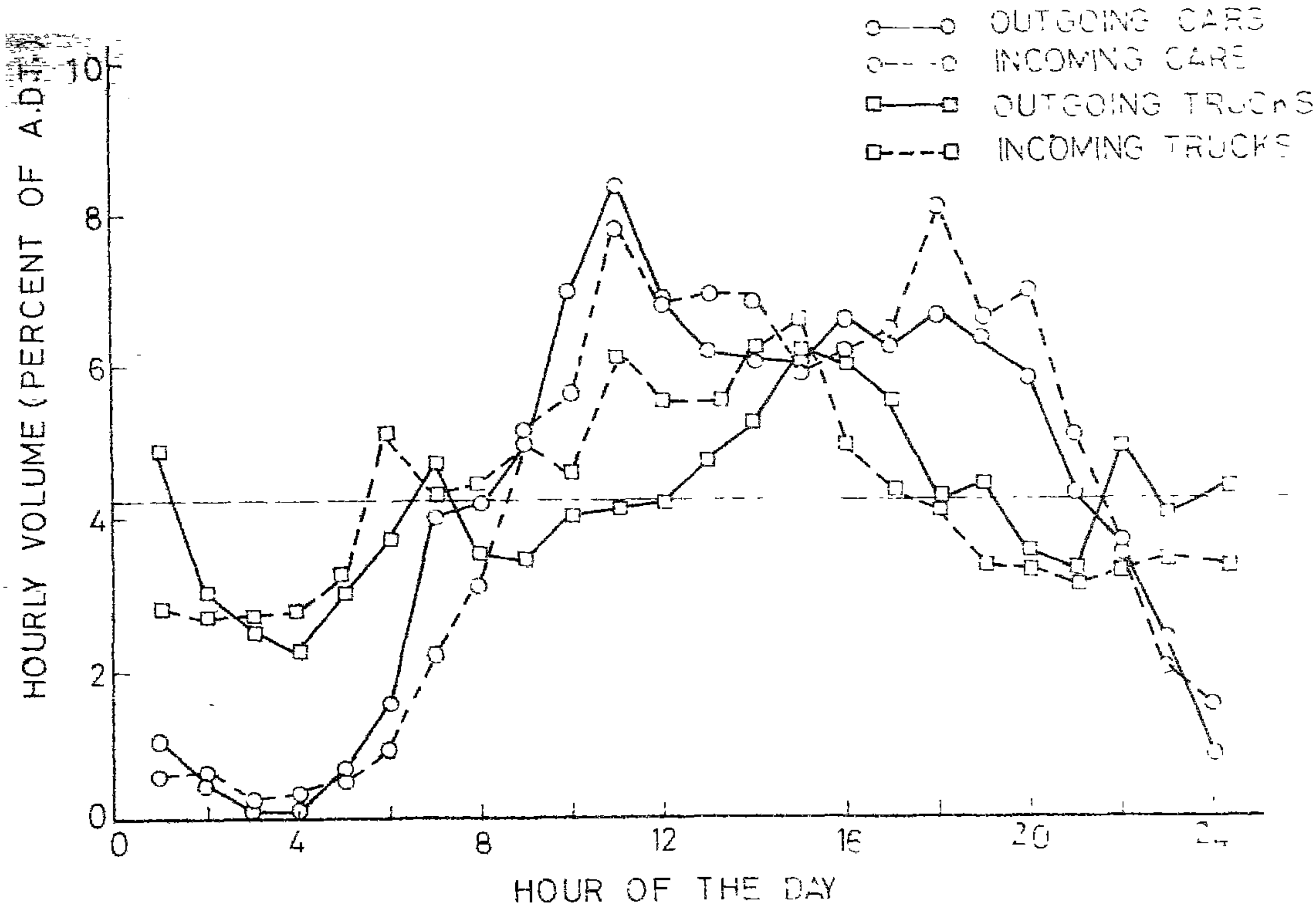


FIG. 3·16 HOURLY VARIATIONS OF TRAFFIC
(CARS AND TRUCKS)

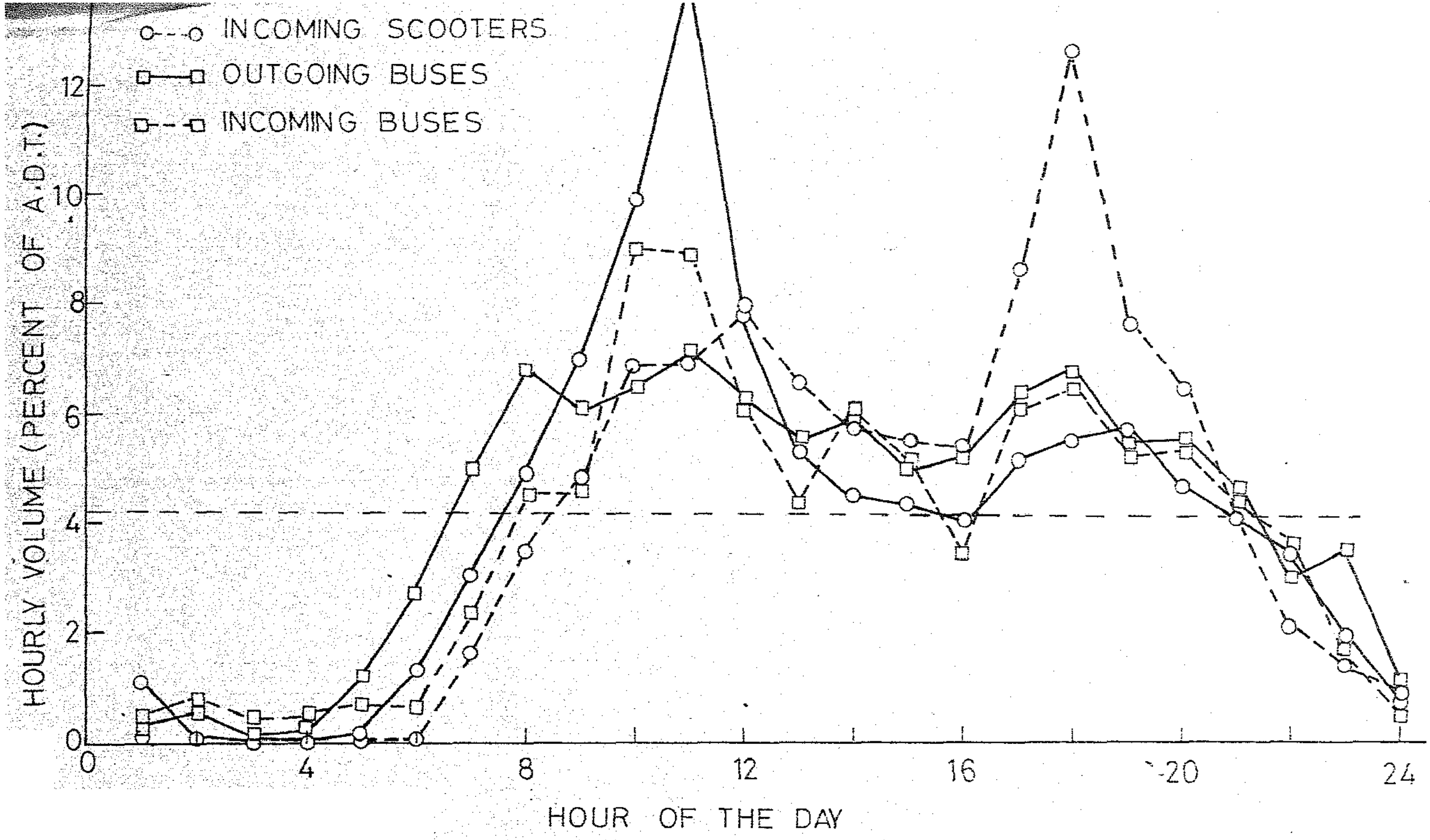


FIG. 3.17 HOURLY VARIATIONS OF TRAFFIC
(BUSES AND SCOOTERS)

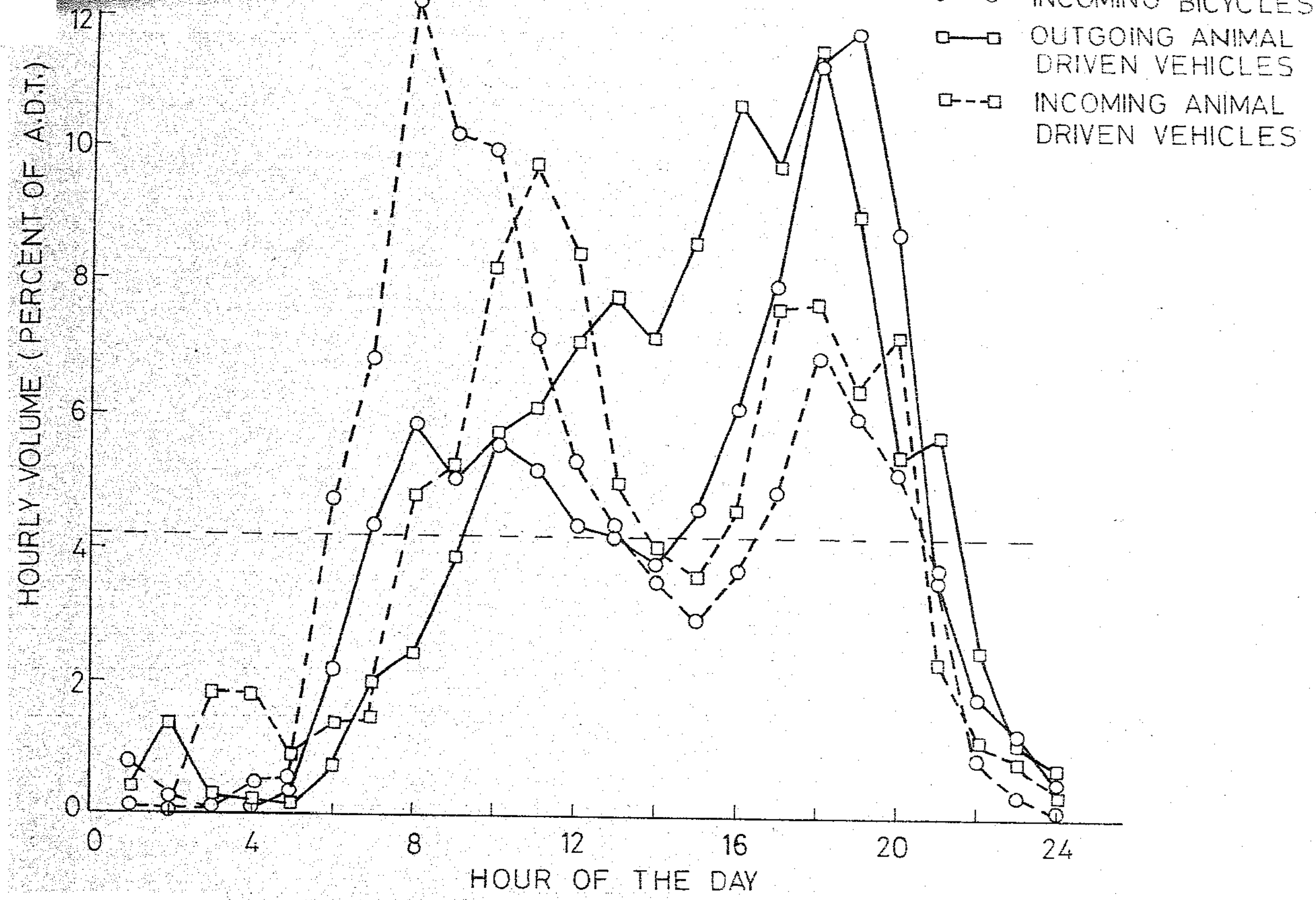


FIG.3.18 HOURLY VARIATIONS OF TRAFFIC (BICYCLES AND ANIMAL DRIVEN VEHICLES)

(1700 to 1800 hours). The peak hourly volume is between 8.0 to 8.5 percent of ADT. It was also observed that 90 percent of ADT travels during 0600 to 2200 hours (16 hours) and the volume is insignificant during the remaining eight hours.

Truck traffic (Fig. 3.16) fluctuates throughout the day with hourly volumes varying generally between 2.2 to 6.5 percent of ADT. Maximum volume is generally in the afternoon between 1400 to 1600 hours for outgoing traffic and between 1300 to 1500 hours for incoming traffic. It was also observed that loaded trucks vary between 78 to 87 percent of the daily truck traffic with a mean of 83 percent and standard deviation of 4 percent. Thus on an average 17 percent of the trucks are empty. Bus traffic (Fig. 3.17) is mostly during the day with 90 percent of ADT between 0600 to 2100 hours. Incoming bus traffic is maximum during 0900 to 1100 hours with maximum hourly volume being 9 percent of ADT. Peaks for the outgoing traffic are around 7 percent of ADT and they occur between 0800 to 1000 hours and 1700 to 1900 hours.

The daily variations of scooters and motorcycles are similar to those of cars . Hourly variations, shown in Fig. 3.17, are quite significant. Maximum hourly volume is as high as 14 percent of ADT and the peaks occur between

1000 to 1100 hours for outgoing traffic and 1700 to 1800 hours for incoming traffic. Also 95 percent of daily traffic moves between 0700 to 2200 hours, Bicycle traffic is minimum on Sunday. Hourly variations (Fig. 3.18) indicate that incoming traffic is maximum between 0700 to 1000 hours and outgoing traffic between 1700 to 1900 hours. Peak hourly volume is about 12 percent of ADT and more than 90 percent of daily traffic travels between 0600 to 2100 hours.

Hourly variations of animal driven vehicles (tongas and bullock carts), shown in Fig.3.18 , indicate that the maximum outgoing traffic moves between 1500 to 1800 hours and is 11.5 percent of ADT; whereas peak for incoming traffic is between 1000 to 1200 hours and is only 10 percent of ADT.

3.6.2 Peak Hourly Composition of Mixed Traffic

Hourly variations of different categories of traffic indicate that the peaks occur at different times for the different categories of incoming and outgoing traffic. For the outgoing traffic maximum volume for cars and scooters is generally in the morning between 1000 to 1200 hours with a smaller peak in the evening. For other categories of vehicles ,the peak is generally in the evening. Thus the critical period for outgoing traffic seems to be between

1600 to 1800 hours in the evening .

For the incoming traffic, peaks for buses, trucks, bicycles and slow moving vehicles occur generally in the morning; whereas the maximum volume for cars and scooters occur in the evening with a smaller peak in the morning. Hence the critical period for incoming traffic seems to be in the morning during 1000 to 1200 hours. The peak hour volumes of different categories of vehicles and the corresponding compositions for the two critical periods are given in Table 3.10.

TABLE 3.10 PEAK HOUR VOLUMES AND COMPOSITIONS

Category of Vehicle	Outcoming ADT	Outcoming Morning (1000-1200) % of ADT	Outcoming Evening* (1600-1800) % of ADT	Incoming ADT	Incoming Morning* (1000-1200) % of ADT	Incoming Evening (1600-1800) % of ADT
Cars	484	8.5	6.6	431	7.8	8.1
Trucks	377	4.5	5.5	380	6.0	4.0
Buses	128	7.0	6.8	121	9.0	6.4
Scooters	352	14.0	6.0	343	7.0	12.5
Bicycles	2341	5.5	12.0	2418	7.0	7.0
Tongas	63	7.0	11.5	61	10.0	7.7
Bullock Carts	47			51		

* Apparantly critical composition

Subsequent studies (Chapter 6) indicated that the PCUS for mixed traffic for both directions are nearly equal during the peak hours.

3.6.3 Estimation of Peak Hourly Volume

Field data and their analyses indicate that the traffic volume varies from month to month within and over the years; from day to day within a month; and from hour to hour within the day. Stochastic models for monthly and daily goods traffic on five roads approaching Kanpur and hourly variations within the day for one road, have been determined. The critical peak hours within the day and compositions of traffic for the critical periods have also been determined. It seems possible to determine the DHV by considering the different variations, e.g., from the monthly and daily variation of traffic, it may be possible to estimate the ADT for different categories of vehicles with a given recurrence interval. From the composition of vehicles at critical periods, the volume of each category of vehicle in the critical period can be determined. When PCES for different categories of vehicles for given volume and composition are known, the equivalent PCUS for mixed traffic in critical periods can be determined. The highways can then be appropriately designed.

4. MATHEMATICAL MODELLING

4.1 Introduction

A dynamic traffic system is characterised by the occurrence of changes in the system variables. A mathematical model consists in representing analytically the system relationships in terms of the parameters of the system and hence representing the changes in the system in analytic terms. The model should represent the system as closely as possible so that the estimates are meaningful. Accuracy of prediction can generally be improved only by making the model more complicated and hence less convenient to use, particularly when a simple model may not be satisfactory. There is hence a necessity for a compromise between accuracy and simplicity (Hall, 1962; Chestnut, 1967).

The advent of digital computers and developments in system analysis and optimisation have radically changed the approach to mathematical modelling of physical systems. While lack of solution techniques was a prime hinderance in the use of sophisticated and more realistic system models, at present the level of simplicity is governed very often by lack of knowledge or understanding of the system.

4.2 Mathematical Models for Traffic Flow

4.2.1 General

The objective of traffic flow modelling is to derive theoretical or empirical relationships between the variables so as to determine the characteristics of the traffic stream. There are three variables of particular interest, viz., flow , density and speed; and they together define the quality of service experienced by the drivers (Wohl and Martin, 1967; Haight, 1963). A complete behavioural understanding of the traffic stream phenomenon is currently not available. Most of the earlier studies were designed to develop workable approximate relationships. In recent years, some of the models based on operations research techniques have also been developed. Some of the deterministic and probabilistic models that describe the stream characteristics of highways are briefly described.

4.2.2 Empirical Methods

The deterministic approach began with a suitable algebraic equation to explain the flow concentration relationships. Greenshields (1934) gave a relationship of the form;

$$\bar{U}_s = \bar{U}_f - \left(\frac{\bar{U}_f}{D_j} \right) D \qquad (4.1)$$

where $\bar{U}_s$ = average space mean speed; $\bar{U}_f$ = mean free speed; D = density; and D_j = jam density.

Greenberg (1958) assumed that high density traffic flow is analogous to continuous fluid flow and based on field studies fitted an exponential function to speed density observations, viz.,

$$D = C e^{b \bar{U}_s} \tag{4.2}$$

where C and b are constants.

A number of empirical relationships based on travel time and volume data have been suggested by Guerin (1961), Normann (1942), etc. They are of the following general type.

$$V = C + b \bar{U}_s \tag{4.3}$$

where V is the volume; and b and C are parameters.

4.2.3 Deterministic Models

Hydrodynamic Analogies: Analogies have been drawn between the flow of fluids and the movement of vehicular traffic. Principal contributions have been made by Greenberg (1958), Lighthill and Whitham (1964) and Richards (1956). The basic assumption is that high density traffic behaves like a continuous fluid. The fundamental equation is;

$$\frac{d \bar{U}_s}{d t} = -\frac{c^2}{D} \frac{\partial D}{\partial x} \tag{4.4}$$

where $\overline{U}_s$ = fluid velocity or space mean speed in kmph; D = density in vehicles per km; x = distance in kms; t = time to travel distance x and c = roadway parameter.

The solution of Eq. 4.3 yields the speed in terms of density as follows:

$$\overline{U}_s = c \ \mathrm{Log}_e \ \frac{D_j}{D} \qquad (4.5)$$

Lighthill and Whitham (1964) have shown that the speed of waves carrying continuous changes of volume through a vehicular flow is given by

$$U_w = \frac{d\ V}{d\ D} = \frac{d\ \overline{U}_s\ D}{d\ D} = U_s + D\ \frac{d\ \overline{U}_s}{d\ D} \qquad (4.6)$$

The interaction between the vehicles is very small at low densities and wave velocity equals the vehicle speed. At densities above the point of maximum volume, the waves will move backward relative to the road and at maximum volume, the wave is stationary relative to the road. Fig.4.1 shows the speed volume density relationships. Due to the changes in wave velocity with vehicular density, it is possible to have different waves traveling through a traffic stream. When traffic moves from a low density road section to a high density section, the waves in the low density traffic will travel forward at a higher speed than the waves in the high density flow. The new wave

formed on the meeting of the two waves is termed as a shock wave (Fig. 4.1). Some of the examples dealing with different conditions occuring in vehicular flow have also been analysed by hydrodynamic analogies (Lighthill and Whitham, 1964).

Car Following Models: A driver probably reacts depending on the distance he is away from the vehicle in front and its rate of acceleration or deceleration. A car following model relates the movement of a single vehicle to the vehicle it follows. Herman and his associates (1959, 1960) at the General Motors Research Laboratories have studied this problem extensively. They assumed that a driver will try to keep the relative speed between his vehicle and that ahead, as small as possible. The mathematical representation is as follows:

$$a_n(t) = K[U_{n-1}(t - T_s) - U_n(t - T_s)] \quad (4.7)$$

where $a_n(t)$ = acceleration of nth vehicle at time t; $U_n(t)$ = velocity of nth vehicle at time t; T_s = time lag or stimulus - response time of driver - car system; and K = driver - sensitivity coefficient.

The driver's sensitivity to the other vehicles increases as the vehicles get closer to one another .

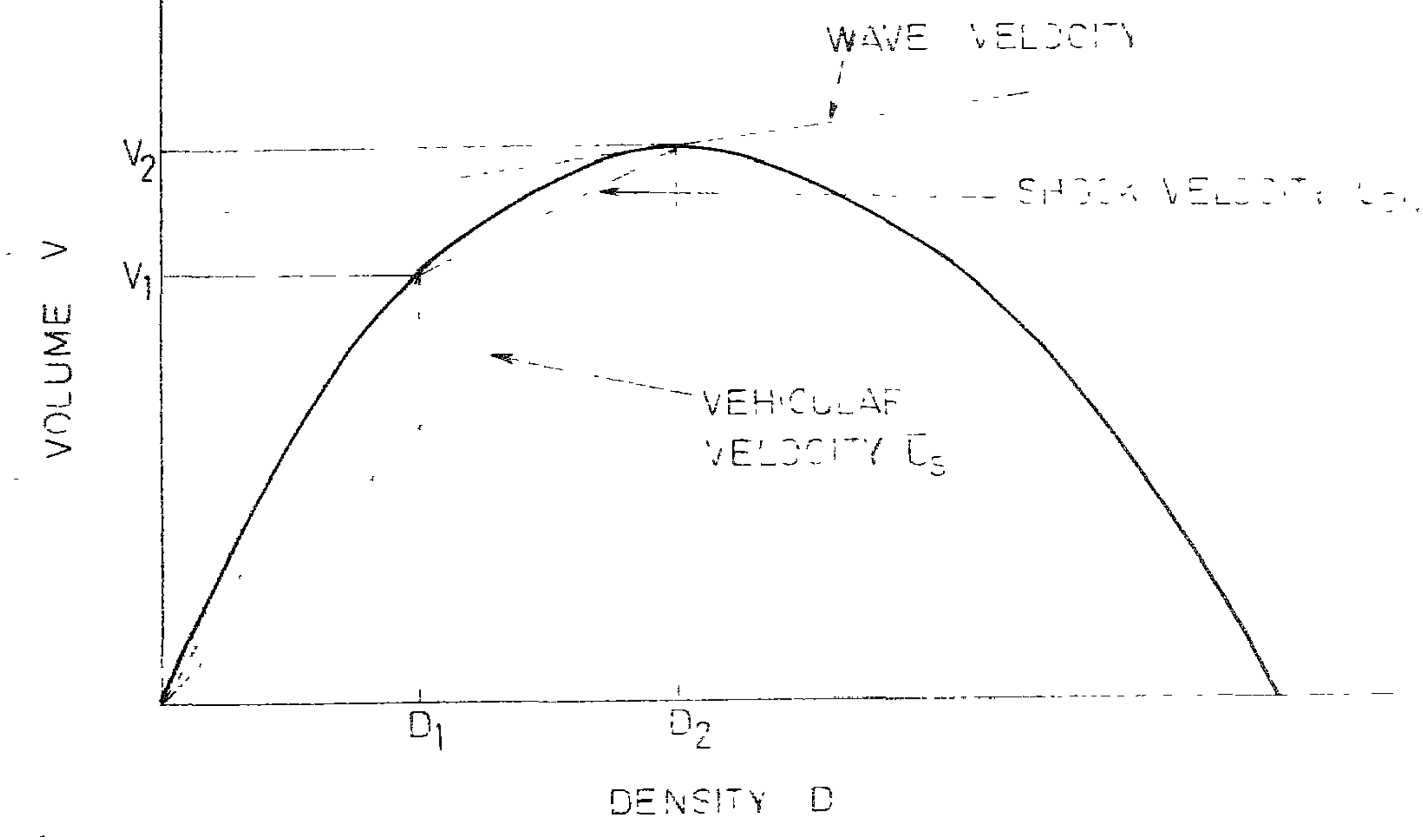


FIG 41 SPEED VOLUME AND DENSITY RELATIONSHIPS

Experiments have also shown that K varies with spacing between the vehicles. Herman (1960) modified Eq. 4.7 as follows:

$$a_n(t) = \frac{K_o}{x_{n-1} - x_n} [U_{n-1} (t - T_s) - U_n(t- T_s)] \qquad (4.8)$$

where K_o = a constant; x_{n-1} and x_n are positions of (n - 1) th and n th vehicles.

The general solution of the above equation indicates that the spacing between successive vehicles has oscillatory characteristics that depend upon the nature of the constant C , defined as

$$C = K_o T_s \qquad (4.9)$$

Edie (1960) has suggested that driver - sensitivity coefficient (K) be taken as a function of both vehicle speed and vehicle spacing, i.e.,

$$a_n(t) = \frac{K_2 U_n(t - T_s)}{(x_{n-1} - x_n)^2} [U_{n-1} (t - T_s) -U_n(t-T_s)] \qquad (4.10)$$

where K_2 is a constant. It leads to the following relationship:

$$D = D_j \text{Log}_e \frac{U_f}{\bar{U}_s} \qquad (4.11)$$

where D_j, the jam density = $\frac{1}{K_2}$

4.2.4 Probabilistic Models

Due to the probabilistic nature of headway gaps, free speeds etc. (Sec. 2.3), traffic flow is probabilistic. Based on queuing theory, flow travel time relationships have been developed by Davidson (1966). He defines the service time as the travel time of a vehicle when no other vehicle is using the road. Queuing delay is taken as the increase in travel time due to the presence of other vehicles on the road. The delay time for a vehicle in queue for random arrival and random service is given by (Cleveland, 1964).

$$d = \frac{a}{u(u-a)} = \frac{c^2}{a(1-c)} \qquad (4.12)$$

where d = the delay per vehicle; a = arrival rate; u = service rate; and c = a/u.

The terms of Eq. 4.12 may be translated into traffic stream variables as follows: a = V = flow, u = S_f = the saturation flow and c = V/S_f. The ratio of the delay(d) in the queue to service time is given by

$$\frac{d}{1/u} = \frac{c^2 u}{a(1-c)} = \frac{c}{1-c} \qquad (4.13)$$

Let t_o = 1/u be the travel time at no flow. Then from Eq. 4.13 , the delay time is given by

$$d = t_o \frac{c}{1-c} \qquad (4.14)$$

Traffic flow is not truly a single continuous queuing situation. Delay is caused by a succession of queuing situations such that a varying amount of the total service time is subject to queuing delays. Davidson (1966) modified Eq. 4.14 to

$$d = t_o J \frac{c}{1 - c} \qquad (4.15)$$

where J is a factor that varies with road type and frequency of delay producing situations along its length. The total travel time t can be expressed as

$$t = t_o \left(1 + J \frac{c}{1 - c} \right) \qquad (4.16)$$

4.2.5 Simulation Models

When suitable conceptual models cannot be formulated for the several components of the process and the complexity is due to the stochastic nature of the process and coherent system components and perheps the presence of multiple alternatives and constraints, computer simulation of the process may be adopted. Actually simulation analysis does not solve any mathematical problem. It consists in setting up of a mathematical model of the system, scanning the process through the system and study the characteristics of the process as it evolves in time (Mize and Cox, 1968).

In simulation, it is possible to vary the parameters of the system, to include stochastic inputs or system responses, to investigate the independence or interdependence of the elements and also, if necessary, to adopt sampling techniques to determine an optimal or near optimal design of the system (Wohl and Martin, 1967).

Analog computers are preferred when the model consists largely of differential equations; if compontents are to be tested separately; frequency of repetition is large; and for 'real-time' and for 'man-machine' system simulation. Digital computers are preferable if the model consists of a large number of arithmatic or logical operations and difference equations and for large size problems (Drew , 1968).

Simulation is hence very suitable for analysis of complex stochastic processes with interacting components. It has been used to a limited extent to study freeway merging operations, interchange design, signalised intersections etc. (Gerlough, 1964) , and are generally limited to the consideration of homogeneous traffic.

4.3 Mathematical Modelling of Mixed Traffic Flow

4.3.1 Complexity of the Problem

Mathematical models described in Sec. 4.2 describe

the flow characteristics of motor vehicles only. The problem of analysing mixed traffic flow is quite complex because of the following factors:

(i) The traffic flow is stochastic in nature and the free speeds of vehicles are also probabilistic.

(ii) There are large variations in the speeds of the fast and slow moving vehicles; thereby increasing the interaction.

(iii) Slow moving vehicles like bullock carts are unaffected by other stream characteristics like volume level and composition and they continue to move at their respective free speeds.

(iv) Vehicles like trucks, cars and bullock carts etc. occupy one full lane, whereas two wheelers like bicycles, scooters etc. occupy very little width and two of such vehicles can very easily move parallel in one lane. Thus the flow logics for two wheelers are significantly different from big vehicles.

(v) The overtaking operations of the fast moving vehicles are restrained in single and double lane roads, due to nonavailability of sufficient gap in the opposite traffic stream.

(vi) The slow moving vehicles have a tendency to move in platoons. A fast moving vehicle has thus to

overtake a full platoon in one operation, thereby needing a lot of time and free gap.

4.3.2 Limitations of Analytic Techniques

Analysis of the mixed vehicular traffic flow by analytic techniques is difficult due to their limitations which include:

(i) Mathematical equations developed from hydrodynamic analogies hold good only for high traffic densities as they rely on the assumption that high density streams of vehicles behave like an incompressible fluid. The character of intervehicle relationship has not been taken into account in the analogy.

(ii) Car following models are generally restricted to the flow of homogeneous vehicles in a single lane with no overtaking. In mixed traffic flow, overtaking is quite important.

(iii) Flow-travel time relationships take into account random distribution of traffic flow. But they are limited to the consideration of homogeneous traffic with constant free speed.

4.3.3 Need for Simulation

Mixed vehicular traffic flow is very complex and it does not seem feasible to use analytic techniques for

studying the process. Yet it is possible to simulate the process on a computer. Because of a large number of logical decisions involved in the flow process, digital simulation is preferred. Computer simulation has the following additional advantages.

(i) It may lead to a better understanding of the nature of the interactions involved including acceleration, retardation, overtaking etc. (Drew, 1968),

(ii) It is useful to validate the component models.

(iii) Data can be generated and used to study the transient and steady state characteristics of the system (Naylor et al., 1966; Mize and Cox; 1968).

Hence it is proposed to use digital computer simulation for the analysis of mixed vehicular traffic.

5. COMPUTER SIMULATION

5.1 General

Digital computer simulation may be defined as, "a numerical technique for conducting experiments with certain types of mathematical models which describe the behaviour of a complex system on a digital computer over extended periods of time" (Naylor et al., 1966). Simulation serves different functions in system analysis and design. They include aids in problem formulation; insight into the sensitivity of design to wide ranges of parameters; guidance in predicting system performance, and validation of system model (Reitmann, 1971).

5.1.1 Steps in Simulation

There are generally a number of steps in simulation of an engineering system. Though the details may vary from time to time, a number of steps are common to several simulation studies. Fig. 5.1 is the representation of the various steps for simulation (Naylor et al. 1966 ; Mize and Cox, 1968).

(I) Formulation of the problem: The problem is formulated in analytic terms. It includes the identification

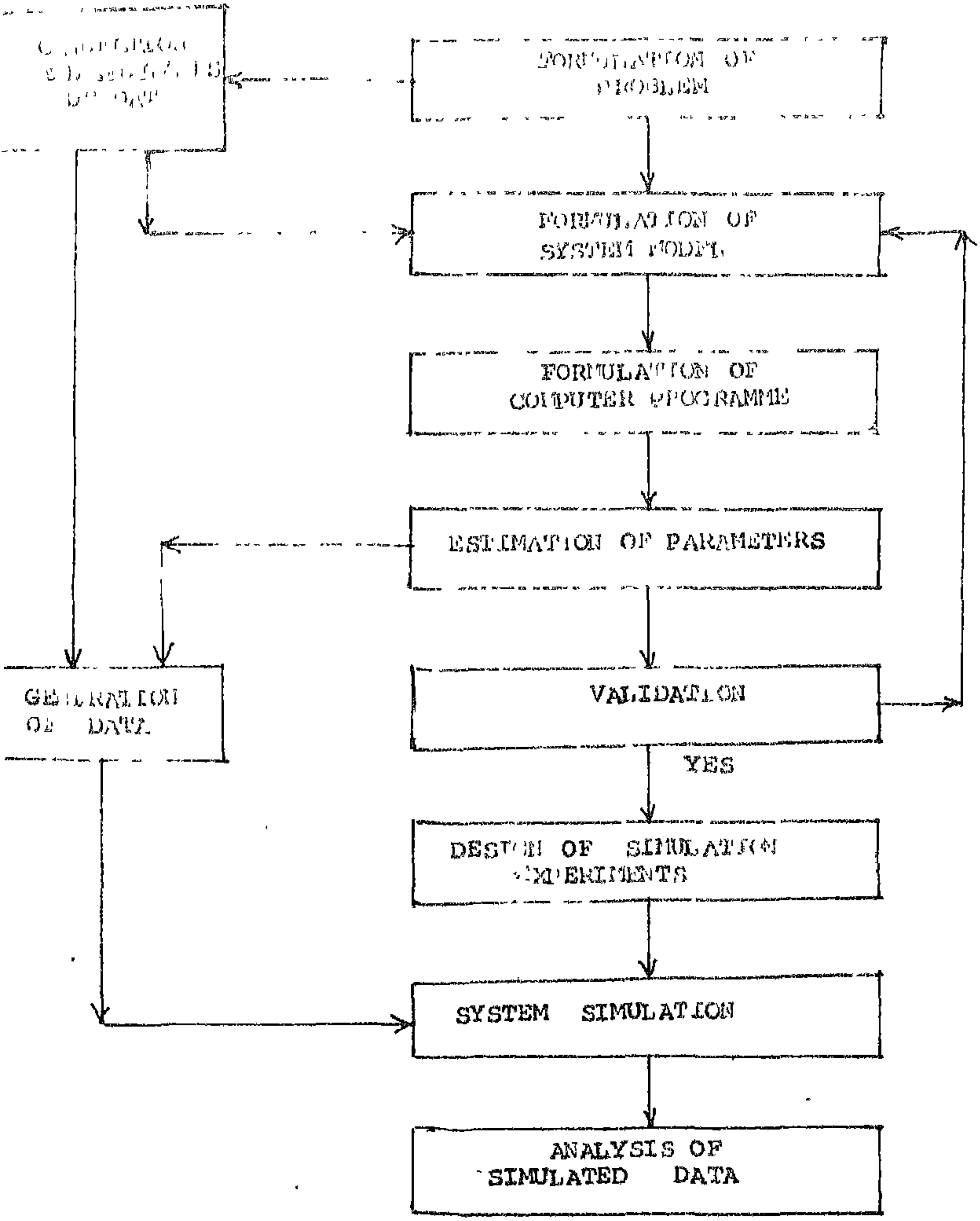


FIG. 5.1 FLOW CHART FOR SIMULATION

of the components of the system and its environment. Constraints on the system are also to be identified.

(II) Collection and analysis of data: Historical and experimental data concerning socioeconomic, engineering and other factors are collected and analysed.

(III) Formulation of the system model: Based on experience, available knowledge and information, a system model is formulated to represent the components of the system, their parameters and behaviour and their relationships; the nature of the inputs to the system, throughputs, and outputs.

(IV) Formulation of a computer programme: The formulation of a computer programme for the purpose of conducting simulation experiments with a model of a complex system requires special considerations to the following three activities; (i) computer programme; (ii) data input and starting conditions; and (iii) data generation.

(V) Estimation of parameters: Parameters of the system are estimated using historical and/or experimental data.

(VI) Validation: The assumed mathematical model for the system is a simple approximation to a more complex reality. It is necessary to validate the assumed system model in order that the mathematical model is a reasonable

approximation to reality. Validation involves the definition of reality and a reasonable approximation to it. Generally validation is done on the basis of comparison between the actual recorded output and the output from simulation model for corresponding inputs. In case the model is considered to be unsatisfactory, it is necessary to modify the model suitably in order that estimated output from the model agrees with the observed record.

(VII) Design of simulation experiments: Simulation may have to be done for analysis and optimisation of complex systems under different conditions of system inputs and outputs, and values of design variables, if any. The values of these variables are generally to be sampled. Hence it may be necessary to select factor levels and combination of factors and the order of experimentation. It is also necessary to ensure that sampling errors are within bounds.

(VIII) System simulation: The system is simulated under different conditions as per the experimental design. When random processes are involved, it may be necessary to generate and use data concerning such variables.

(IX) Analysis of simulated data: Results from simulation are analysed to determine the characteristics of various components of the process, their significance

and interdependence. A near optimal design may be obtained in appropriate cases.

5.2 Model Formulation

5.2.1 System Description

The details of the system involving various components, variables, relationships and assumptions involved in the mathematical modelling of system simulation are discussed briefly.

System Components: A graphical representation of the roadway section including the various system components is shown in Fig. 5.2. The components include:

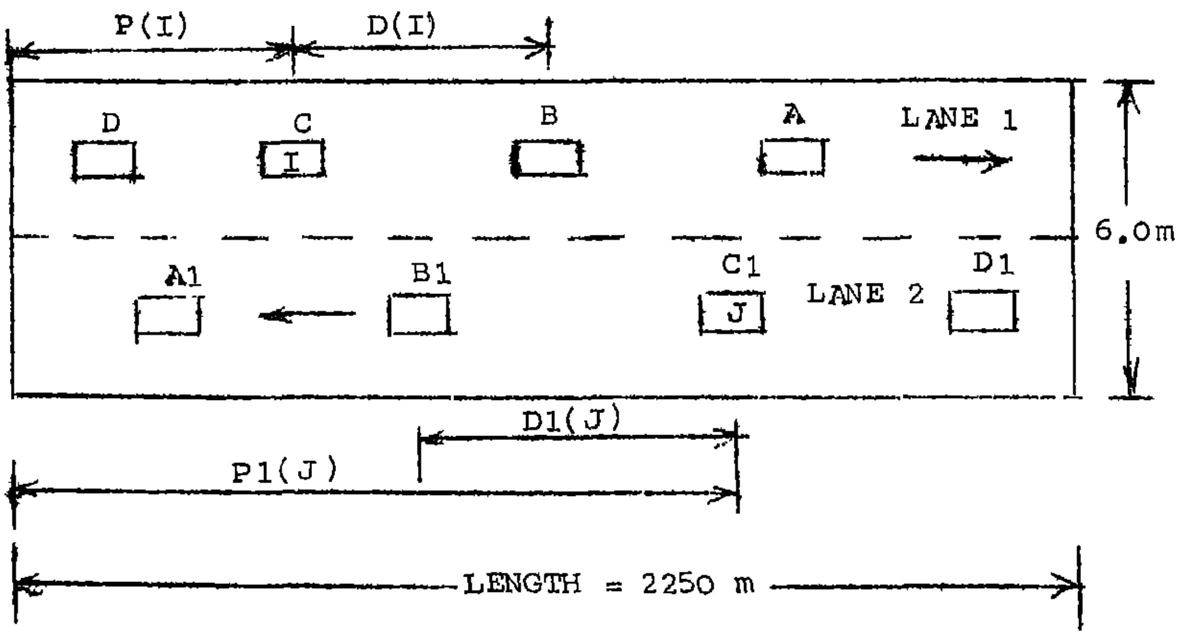


FIG. 5.2 ROADWAY SECTION AND VARIOUS COMPONENTS

(i) Length of roadway section - 2.5 km.

(ii) Width - 2 lane , 6.0 metre wide cement concrete pavement with 1.5 metre wide shoulders on each side.

(iii) Lateral obstructions - no natural or man made obstructions are present to restrict free flow.

(iv) Horizontal alignment - the section is a straight reach with no sight distance restrictions due to bends etc.

(v) Vertical alignment - the longitudinal gradient is quite flat and does not affect the free flow.

(vi) Presence of intersections and other regulatory and control measures. The section under study was free from all such restrictions.

Assumptions: The following general assumptions have been made in model formulations:

(i) The roadway section was considered to be sufficiently away from traffic control points, e.g., intersections at both ends. Vehicles were assumed to approach the section in a random fashion. The section selected for field studies generally satisfied this requirement.

(ii) Vehicles approach the section from both directions at their free speeds. The free speed of a certain category of vehicle is probabilistic and perhaps also a

function of time and space. However, the free speed distribution parameters were assumed to be constants with no variations in time and space.

System Variables: Variables are those attributes of the system that take on different values under different conditions, or in different system states. They can be classified into several ways as exogeneous, endogeneous, status and random variables, etc. An exogeneous (or input) variable (Naylor et al., 1971; Mize and Cox , 1968) is one that is quantifiable and which affects, but is itself unaffected by the system. Status variable is one that represents the state of an important characteristic of the system, which is generally a throughput variable. A variable having a value determined by other variables in the system, is an endogeneous variable (Naylor et al., 1971) and represents in a way the output of the system. The following variables may be identified in the traffic system under study:

Exogeneous Variables: They include:

(i) Interarrival time gaps AG(I) and AG1(J) of the vehicles approaching from either direction. The vehicles are to be identified and processed simultaneously from two directions. So separate names were given to variables for either direction with the letter 1 before brackets identifying

the characteristics of vehicles in lane 2. Subscript I pertains to the serial number of vehicle in lane 1 and J to that in lane 2. Interarrival time gaps are used to compute the arrival times AT(I) and AT1(J);

(ii) Hourly volume of traffic NX and NX1 and the number of categories of vehicles in the mix NCAT, NCAT1 alongwith their proportions PROB(K) and PROB1(K) for K = 1 , NCAT (NCAT1) ;

(iii) Category of the vehicle entering the section from either direction ICAT(I) and ICAT1(J) . ICAT and ICAT1 are defined by codes 1-6 as follows:

ICAT(I) or ICAT1(J) = 1 for cars; 2 for trucks; 3 for bullock carts; 4 for tongas; 5 for scooters; and 6 for bicycles; and

(iv) Free speed of arriving vehicles, VF(I) and VF1(J). Free speeds follow a censored normal distribution for each category of vehicle.

Status Variables: They include:

(i) Position of vehicles P(I) and P1(J) along with their running speeds V(I) and V1(J) at any instant;

(ii) Available headways D(I) and D1(J) for each vehicle at any instant;

(iii) Overtaking time OT(I) and OT1(J) in case a vehicle is overtaking at any instant. Otherwise OT(I) and

OT1(J) are equal to zero;

(iv) Traffic stream logic for the vehicle at any instant i.e., NVT(I) and NVT1(J) . Different stream logics are indicated by codes; and

(v) Number of vehicles in the section i.e., density of each category IQE(K) and IQE1(K) for K = 1, NCAT (NCAT1) at any instant.

Endogeneous Variables: They include:

(i) Actual departure time DEPT(I) and DEPT1(J) of the vehicles from the system. These are used to compute delay times for each vehicle DELT(I) and DELT1(J) ;

(ii) Number of vehicles of each category going out of the section NDPT(K) and NDPT1(K) ;

(iii) Operating speed of the departed vehicles VEF(I) and VEF1(J) ;

(iv) Number of vehicles delayed NDEL(K) and NDEL1(K) in each category; and

(v) Average operating speed AVVE(K) and AVVE1(K) for each category.

Random Variables: The following three variables are probabilistic in nature (Sec. 2.3):

(i) Arrivals are random and interarrival time gaps AG(I) and AG1(J) follow a shifted exponential distribution;

(ii) Category of the arriving vehicles ICAT(I) and ICAT1(J). The probability that arriving vehicle belongs to a specific category is considered to be proportional to the proportion of the vehicles of the same category in the overall traffic composition; and

(iii) Free speed of vehicles VF(I) and VF1(J). The free speeds of a particular category follow a censored normal distribution.

System Relationships: Some of the general relationships between the variables of the mathematical model are given below. Relationships that are more specific to the simulation analysis are given subsequently in Sec. 5.4. Equations are given for the two lanes separately only where necessary. Otherwise relationship for lane 1 is given and a similar relationship holds good for lane 2.

(i) Available headway for a vehicle at any instant, i.e., D(I) and D1(J). The vehicles from either direction are arranged in the order of their positions L and L1 and KK(L) = I and KK1(L1) = j. The available headways can be calculated as follows:

$$D(I) = P(KK(L-1)) - P(I) \tag{5.1}$$

and

$$D1(J) = P1(J) - P1(KK1(L1-1)) \tag{5.2}$$

(iii) Expected departure time of the vehicles, i.e., EDEPT(I) and EDEPT1(J), if they continue to move unimpeded

at their free speeds throughout the length of the section, i.e., there is absolutely no delay.

$$EDEPT(I) = AT(I) + \frac{LNGT}{IX} \qquad (5.3)$$

where AT(I) = arrival time of I th vehicle in lane 1;

LNGT = length of the section; and

IX = distance covered by Ith vehicle in one second at free speed VF(I) in kmph

= 0.28 VF(I)

(iii) Delay time of a vehicle DELT(I) and DELT1(J).

$$DELT(I) = DEPT(I) - EDEPT(I) \qquad (5.4)$$

(iv) Operating speed of the vehicle in kmph

$$VEF(I) = \frac{LNGT}{0.28\ (DEPT(I) - AT(I))} \qquad (5.5)$$

(v) Average operating speed of each category of vehicle AVVE(K) and AVVE1(K) and their standard ~~derivations~~ deviations SDVEF(K) and SDVEF1(K).

$$AVVE(K) = \frac{1}{NDPT(K)} \sum_{I=1}^{NDPT(K)} VEF(I) \qquad (5.6)$$

$$SDVEF(K) = \sqrt{\frac{\Sigma(VEF(I))^2 - NDPT(K)(AVVE(K))^2}{NDPT(K) - 1}} \qquad (5.7)$$

5.2.2 Scanning Techniques

The process is simulated along time and scanned or observed at intervals. The computer programme must be

written in such a way that it moves the model over simulated time, causing events to occur in proper order and with a proper time interval between successive occurrences of events.

The real components function simultaneously whereas the components are simulated sequentially. There are dependencies among different parts of the real system. It is important that a time flow mechanism be embedded and the process scanned so that simulated performances of the system components are synchronised in time. This can be accomplished in two ways (Blake and Gordon, 1964) ; (i) periodic scanning or uniform increment method; and (ii) event oriented scanning or variable increment method.

In periodic scan method, the duration of the simulation run is divided into a number of equal time intervals, that is, scan and update the entire system (like move vehicles, change speeds etc.) once per unit time interval. In the event scan method, when a specific event has occured, the status of a system is updated and among the possible events that can occur, a scanning is done to determine when the next event occurs and the system is again updated. The procedure is repeated. Hence event scanning essentially deals with the scanning of the process at nonuniform time intervals depending upon the occurrence

of the next event; while periodic scanning is done at uniform time intervals. Event scan technique is faster and more accurate due to less variance of estimates, but it usually requires greater programming complexity. Periodic scanning is more straight forward and easy to programme.

The traffic flow process requires scanning both along time and space in order to determine the position and other characteristics of vehicles at different times. Continuous scanning in time and space is not possible in digital simulation and so time and spacing were discretised. The roadway space is divided into one metre long one lane wide blocks. The vehicles are moved into the system based on their own characteristics and also their interactions with other vehicles in the system. At different times, a vehicle may be accelerating, running at free speed, overtaking or decelerating depending on traffic conditions. With so many vehicles in the system having varying stream logics (Sec. 5.3) at different times, it is very difficult to identify the immediate next event. In such a case where vehicular characteristics like speed etc., are time dependent, periodic scanning is easy to formulate and was adopted in this study. Scanning was done uniformly at one second intervals based on the vehicular characteristics at that instant, and vehicles were stepped up one metre long

one lane wide blocks.

New vehicles are brought into the system with arrival gaps following a certain probability distribution. Arrival times of all the vehicles likely to enter during next specified simulation period from either directions are generated and stored. Simulation starts with periodic scanning and at each time, it is tested whether a vehicle is likely to enter from any direction or not. If a vehicle enters, it is subsequently moved and scanned periodically. For overtaking vehicles, the overtaking time and the position of vehicle on completing the operation are computed. Hence for the overtaking vehicle, scanning is not needed for this duration. But on completion of overtaking operation, the vehicle is brought back into the system for periodic scanning.

5.2.3 Computer Representation of the Simulation Model

Digital simulation deals with studying the process as it takes place through the system. This is accomplished by representing the components or their characteristics and the variables in the computer and manipulating them as the process takes place along time. Mixed traffic flow on the two lane highway involves the representation and manipulation of the vehicles over the two lanes. There are two general procedures for such representation (Gerlough , 1956, 58;

Wohl and Martin, 1967), namely, (i) Memorandum representation and (ii) Physical representation.

In memorandum method, the characteristics of the vehicles like category, speed and position etc. are indicated by using one or more coded words, generally one word per characteristic. In each time interval, the data are updated and the imformation concerning the characteristics is changed, if and when necessary.

In physical representation, the memory locations are associated with blocks in physical space. The information in the memory location characterises the information concerning the presence or absence of the vehicle in the given block and their characteristics. The data in the memory locations are scanned to identify the characteristics of the traffic and are updated in every time interval.

There are 4500 one metre long one lane wide blocks to represent the space of the roadway section. Further more, there are large number of characteristics of an individual vehicle with wide variations in speeds and other characteristics. Because of all these factors, memory requirement is very large for physical representation. In addition, data manipulation is also very complicated. Hence in this study, it was decided to adopt memorandum representation of the simulation process. The vehicles were identified individually;

their characteristics were stored separately; and were updated and analysed in each time interval.

5.3 Traffic Stream Logic

5.3.1 Introduction

The logic that the vehicles follow as they move through the system is referred to as traffic stream logic (Drew, 1968). The logic for any vehicle depends upon its position and speed relative to other vehicles moving in both directions and as the process is dynamic, it will change from time to time. The stream logic was classified into the following three categories for this study:

(i) Flow logic for unimpeded vehicles, i.e., vehicles that continue to move at their free speeds;

(ii) Flow logic for overtaking vehicles that decide to overtake; and

(iii) Flow logic for restrained vehicles i.e., for vehicles that desire to overtake; but are restrained due to opposite traffic stream.

5.3.2 Spacings for Vehicles

A vehicle has to keep a certain minimum spacing with the vehicle ahead of it so as to avoid collision if the forward vehicle suddenly decelerates or stops. Spacing

is defined in this study as the distance measured from centre to centre of successive vehicles. The minimum spacing is generally a function of driver reaction time, speed and length of vehicle, coefficient of friction, etc. A certain minimum spacing is also needed when a vehicle overtakes another vehicle and reenters its own lane. The spacing will govern the position of the vehicle after overtaking and hence the overtaking time.

When the vehicle ahead is moving faster than the rear vehicle, headway available to the rear vehicle continues to increase. However, if the rear vehicle is moving faster, then it will either try to keep the minimum spacing or else overtake. The decision to overtake is assumed to be taken when the available headway is equal to or less than the specified maximum spacing for overtaking. Based on empirical field observations and available specifications, these spacings are to be specified.

Minimum Spacing for Vehicles: The minimum spacing for vehicles is a function of speed and length of vehicles. The following equation (IRC, 1950) was adopted in this study.

$$S(I) = 0.2\ V(I) + L_o \qquad (5.8)$$

where $S(I)$ = minimum spacing; $V(I)$ = speed of vehicle I; and L_o = constant which varies with the category of vehicles

and may be a function of length of vehicle I and of that ahead of it.

Spacing for Overtaking Vehicles: A vehicle desiring to overtake needs more spacing to permit the driver to manoeuvre the vehicle to the other lane. Maximum spacing for overtaking is a function of the speed of vehicle and also the drivers reaction time for taking the decision. This spacing is thus defined as the distance that can be covered by the vehicle during the headway time of T_o seconds, viz.,

$$SD(I) = v(i).\ T_o$$

$$= 0.28\ V(I).\ T_o \qquad (5.9)$$

where V(I) is the speed of the overtaking vehicle in kmph and T_o is the headway time in seconds which is a parameter.

For slow moving vehicles, the maximum spacing for overtaking SD(I), calculated on the basis of above equation may be smaller than the minimum spacing S(I), in which case SD(I) was limited to S(I). This was the case when a tonga or bicycle overtakes a bullock cart.

5.3.3 Flow Logic for Unimpeded Vehicles

Let D(I) be the available spacing between the vehicle I and that ahead of it at any instant (Fig. 5.2 and Eqs. 5.1 and 5.2. The available spacings at an instant can be

computed by arranging all the vehicles moving in a lane in the order of their positions. When the free speed of a vehicle is less than that of the vehicle ahead, the rear vehicle can continue to move unimpeded irrespective of the available spacing which is increasing with time. When the available spacing D(I) for a vehicle at an instant is more than the maximum spacing needed for overtaking i.e., $D(I) > SD(I)$; then also this vehicle can continue to move unimpeded at its free speed in its own lane. Otherwise the speed of the vehicle will be affected by traffic conditions.

5.3.4 Overtaking Logic

It may be noted that L gives the ordered position of the vehicle in a lane, KK(L) gives the sequence number of the vehicle based on arrival times, i.e., $KK(L) = I$ and $KK1(L1) = J$.

When the speed of a vehicle is more than that of the vehicle ahead, i.e., $V(KK(L)) > V(KK(L-1))$ and the available spacing D (KK(L)) is less than or equal to the maximum spacing for overtaking, ie., $D(KK(L)) \leq SD(KK(L))$; then the vehicle has a tendency to overtake ;

(i) When the speed difference between two successive vehicles is less than 16 kmph, i.e.,

$$V(KK(L-1)) < V(KK(L)) < V(KK(L-1)) + 16 ,$$

the rear vehicle can overtake and the following assumptions concerning the overtaking operations are made.

The rear vehicle slows down to the speed of the vehicle ahead and follows it in its own lane for a time T_R, when the driver looks in the opposite stream for a gap sufficient enough for overtaking. If overtaking is possible, the vehicle accelerates, goes to the wrong lane and moves back to its own lane after overtaking. The rate of acceleration varies with speed and Indian Roads Congress (IRC, 1950) specifications for motor vehicles given in Table 5.1 are used. Fig. 5.3 shows the details of the overtaking operation.

TABLE 5.1 MAXIMUM OVERTAKING ACCELERATIONS AT DIFFERENT SPEEDS

Speed kmph	Maximum Overtaking Acceleration kmph/sec
25	5.00
30	4.80
40	4.45
50	4.00
65	3.28
80	2.56
100	1.92

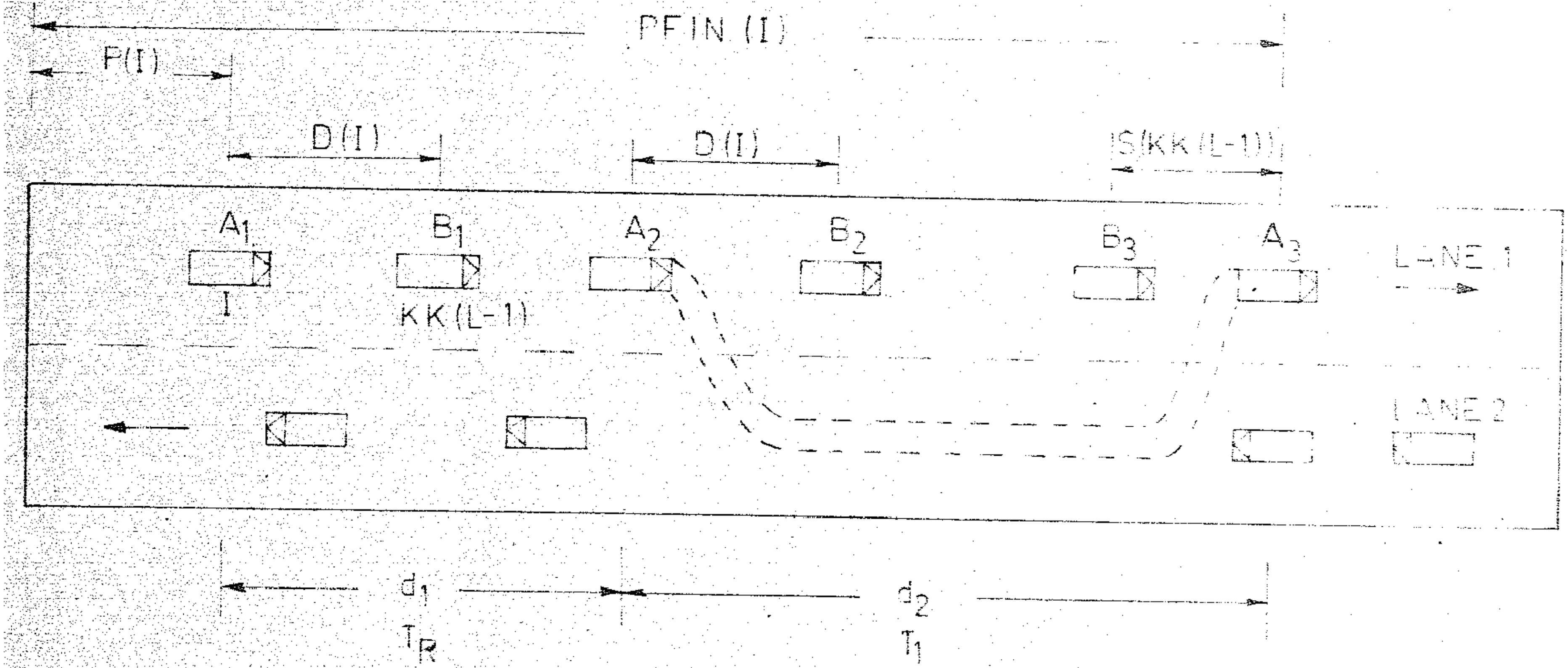


FIG.5.3 OVERTAKING OPERATION

Let A_1 and B_1 represent the positions of two successive vehicles at any time with $[V(KK(L-1)) \lesseqgtr V(I) < V(KK(L-1) + 16]$ and the available headway $D(I) < SD(I)$, then the vehicle I (= KK(L)) reduces its speed to that of vehicle KK(L-1) and the vehicles move from A_1 to A_2 and and B_1 to B_2 respectively during the looking for gap time T_R. If overtaking is possible, the vehicle I starts accelerating from A_2 , goes to the other lane and comes back to its own lane at A_3. During the same period (T_1) the vehicle KK(L-1) moves from B_2 to B_3. The overtaking vehicle occupies the wrong lane for a distance d_2 given by

$$d_2 = d + D(I) + S\ (KK(L-1)) \qquad (5.10)$$

where d = distance travelled by the vehicle KK(L-1) during time $T_1 = [0.28\ V(KK(L-1)).\ T_1]$; D(I) = the available headway for overtaking vehicle I; and S(KK(L-1)) = minimum spacing for vehicle being overtaken $= [0.2V\ (KK(L-1)) + L_o]$. All the distances are measured longitudinally and the lateral movement of vehicles to the wrong lane is ignored .

The overtaking vehicle(I) is travelling at speed V(KK(L-1)) from A_1 to A_2 and accelerates from A_2 to A_3. The distance d_2 can be represented as

$$d_2 = 0.28\ V\ (KK(L-1)).T_1 + \frac{1}{2}\ \alpha T_1^2 \qquad (5.11)$$

where α is the acceleration in kmph/sec.

From Eqs. 5.10 and 5.11

$$\frac{1}{2}\alpha T_1^2 = D(I) + S(KK(L-1))$$

and

$$T_1 = \sqrt{\frac{2(D(I) + S(KK(L-1)))}{\alpha}} \quad (5.12)$$

The total overtaking time needed by the vehicle to move from A_1 to A_3 is

$$OT(I) = T_R + T_1 \quad (5.13)$$

and the final position (A_3) of the overtaking vehicle can be given by

$$PFIN(I) = P(I) + D(I) + 0.28\ V(KK(L-1))\ OT(I) + S(KK(L-1)) \quad (5.14)$$

(ii) When the speed difference between two successive vehicles is more than 16 kmph (as in case of motor vehicle following a slow moving vehicle) , overtaking vehicle is assumed to travel at its normal free speed. During the looking for gap time (T_R), the vehicle generally decelerates and subsequently accelerates to its normal speed and hence covers a smaller distance. It is assumed that the distance lost in acceleration and deceleration is equal to $[0.28\ T_R\ (V(I) - V(KK(L-1))]$. Hence the relative distance covered by vehicle during overtaking operation is given by;

$$A = D(I) + S\ (KK(L-1)) \tag{5.15}$$

This distance is travelled at a relative speed of

$VV = V(I) - V\ (KK(L-1))$ during time

$$T_1 = A/0.28\ VV \tag{5.16}$$

The total overtaking time is given by $OT(I) = T_R + T_1$.

The overtaking vehicle occupies the wrong lane while moving from A_2 to A_3 for time T_1. Overtaking is possible only if there is no conflict between the overtaking vehicle and the traffic stream coming from opposite direction. All the vehicles of the opposite stream are moved in their own lane for T_1 seconds to check whether any vehicle lies between A_2 to A_3. After the check, vehicles of opposite stream are moved back to their original positions. If there is no liklihood of any conflict, the vehicle I moves to the wrong lane and comes back after OT(I) seconds to position PFIN(I).

Sometimes when a fast moving vehicle finds a platoon of slow moving vehicles moving ahead of it, and the available gap between any two vehicles of the platoon is not sufficient for it to reenter, then it desires to overtake the platoon in one operation. Fig. 5.4 shows the overtaking of a platoon of vehicles.

Let the vehicle I (= KK(L)) follow the vehicles KK (L-1), KK(L-2) , KK(L-3), etc. When vehicle I is at

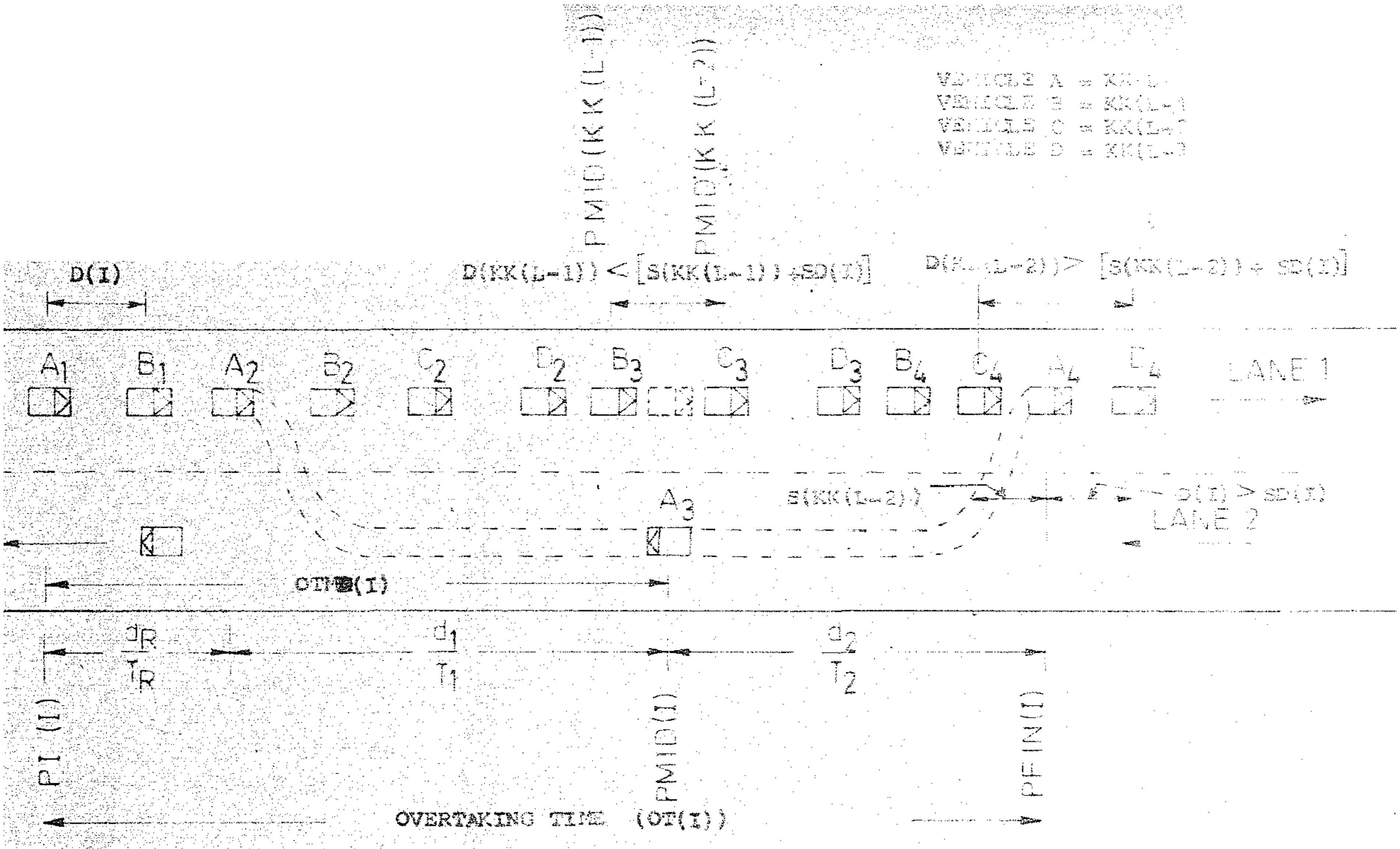


FIG. 5.4 OVERTAKING OF A PLATOON OF VEHICLES

A_1 and $V(I) > V(KK(L-1))$, it desires to overtake KK(L-1). The time to overtake KK(L-1) is ;

$$OTMD(I) = T_R + T_1 \quad (5.17)$$

where T_1 is the time during which vehicle KK(L-1) moves from B_2 to B_3, i.e., PMID (KK(L-1)) and vehicle KK(L-2) from C_2 to C_3, i.e., PMID(KK(L-2)). The available gap for (KK(L-1)) is;

$$D(KK(L-1)) = PMID(KK(L-2)) - PMID(KK(L-1)) \quad (5.18)$$

If D(KK(L-1)) is less than (S(KK(L-1)) + SD(I)), then the gap is not enough for vehicle I to enter between KK(L-1) and KK(L-2) . If $V(I) > V(KK(L-2))$, the vehicle I desires to overtake KK(L-2) also. When vehicle KK(L-1) is at B_3, the position of vehicle I at A_3 is given by;

$$PMID(I) = [P(I) + 0.28\ V(KK(L-1)) \cdot OTMD(I) + D(I) + S(KK(L-1))] \quad (5.19)$$

and $$D(I) = D(KK(L-1)) - S(KK(L-1)) \quad (5.20).$$

The time to overtake KK(L-2) is T_2 during which vehicle KK(L-2) and KK(L-3) move from C_3 to C_4 and D_3 to D_4 respectively. When KK(L-2) is at C_4 and the available gap $D(KK(L-2)) > [SD(I) + S(KK(L-2))]$, the vehicle I can move between KK(L-2) and KK(L-3) to position A_4. Otherwise I will overtake KK(L-3) also. Let T_2 be the time taken by vehicle I to move from A_3 to A_4. Vehicle I enters its own lane at

position A_4 given by ;

$$PFIN(I) = PMID(I) + D(I) + 0.28\ V(KK(L-2))\ T_2 + S(KK(L-2)) \qquad (5.21)$$

after a total overtaking time of

$$OT(I) = OTMD(I) + T_2 \qquad (5.22)$$

The above procedure can be adopted for calculating overtaking time and final position after passing a number of platooned vehicles.

5.3.5 Flow Logic for Constrained Vehicles

Overtaking is not possible when the vehicle desiring to overtake is likely to come in conflict with the opposite traffic stream. For the restrained vehicle to move in its own lane, a minimum spacing must be maintained. It can be done only if the vehicle reduces its speed to that of vehicle ahead, i.e. V(I) = V(KK(L-1)). The vehicle I continues to move at this speed till it is able to overtake and maintain its own free speed. Such constraints delay the vehicles thereby reducing the operating speeds.

5.4 Simulation Logic

Simulation starts at time T = 0 second. The vehicles entering the section from each direction are identified by their sequence numbers and data concerning each of these

vehicles including arrival time, category and free speed etc. are stored by the memorandum process. Initially the lanes are assumed to be empty and at each unit time interval (1 second) , the vehicles are moved according to their stream logic defined in Sec. 5.3. To accomplish this, the following procedure was adopted.

All the vehicles moving in a lane, including new arrivals and excluding overtaking vehicles at that instant, are arranged in the order of their positions. In one of the lanes, the arranged vehicles are scanned from the vehicle at the head of the stream. The available headway and the speed difference between successive vehicles are calculated for all other vehicles in the stream. The stream logic is determined for each vehicle. For any vehicle desiring to overtake, it is checked as to whether opposite traffic comes in conflict with the overtaking operation or not. This is done by moving the opposing vehicles on an adhoc basis for the duration of overtaking operation at normal speeds. If there is no conflict, the vehicle can perform overtaking and be at the predetermined position at the end of its overtaking times. It is hence not included in the consideration of stream logic for the duration of overtaking operation. It is considered again from the moment it reenters its own lane. In case of conflict, overtaking is not possible and

the vehicle is restrained to move in its own lane at the speed of the vehicle ahead. Scanning for a unit interval is complete when the last vehicle of the stream has been considered. Scanning is similarly done for the other lane. The vehicles in both the lanes are moved ahead for one second interval at the appropriate running speeds determined from stream logic.

The above procedure is repeated for each time interval. Vehicles going out of the section are taken out of the system and their characteristics like departure time, delay time and operating speed,etc., are computed and stored. Simulation is stopped at an appropriate time and group characteristics are computed for each category of vehicles.

5.5 Data for Simulation

5.5.1 Need

Data concerning the several characteristics and the inputs of process are required for computer simulation. In case adequate and representative data are available, they may be read into the computer. Otherwise, it may be necessary to generate one or more sets of representative data in the computer for subsequent use in simulation. In

such a case, the problem is one of random selection from the probability distribution of the exogeneous stochastic variable.

5.5.2 Pseudorandom Number Generator

Uniformly distributed random numbers play an important role in the generation of random variables drawn from other probability distributions. The numbers generated by computer subroutines are called pseudorandom numbers, as they are generated by a completely deterministic relationship and yet their statistical properties agree with those of an idealised chance device (Naylor et al., 1971).

An ideal pseudorandom number generator should yield sequence of numbers that are uniformly distributed; statistically independent; reproducible ; and nonrepeating for any desired length. Further such a generator should be capable of generating random numbers at high speed and require minimum amount of computer memory. Congruential methods (Rotenberg, 1960 ; Naylor et al., 1966) are generally used for generating pseudorandom numbers in a digital computer.

Multiplicative congruential method was used for generation and it is given by ;

$$\left.\begin{aligned} n_{i+1} &= \lambda\, n_i \pmod{m} \\ \text{and} \quad r_{i+1} &= \frac{n_{i+1}}{m} \end{aligned}\right\} \qquad (5.23)$$

where $\lambda = 5^5$ and modulus $m = 2^{35}-1$. Starting number n_o was taken to be an odd number. By changing the initial value n_o, 16 different sequences of uniformly distributed random numbers were generated. The generated sequences were found to be uncorrelated and uniformly distributed.

5.5.3 Random Numbers with Specified Distribution

Generation of random variables of different distributions can be derived from uniformly distributed random variables. Let the cumulative probability function $F(X)$, of a stochastic process be

$$F(X) = \text{Prob}\,[x \leqslant X] = \int_{-\infty}^{X} f(t)\, dt \qquad (5.24)$$

where $F(X)$ is defined over the range $0 \leqslant F(X) \leqslant 1$ and $f(t)$ represents the probability density function of random variable X at $X = t$.

As $F(X)$ has a uniform distribution over (0-1) range, uniformly distributed random numbers r in the (0-1) range are generated and set $F(X) = r$. For any particular value of r (say r_o), which is generated, it is possible to find the value of X (in this case X_o) corresponding to r_o by the

inverse function(Naylor et al, 1966).

$$X_o = F^{-1}(r_o) \qquad (5.25)$$

where $F^{-1}(r_o)$ is the inverse transformation of r on the unit interval into the domain of X.

5.5.4 Data Generation for Input Variables

The following three variables are probabilistic whose data are to be generated:

(i) arrival time or intcrarrival time gap between consecutive vehicles entering from either direction of the road section;

(ii) category of the arriving vehicles like car, truck, bullock cart, bicycle etc.; and

(iii) free speed of the arriving vehicles.

Vehicular Arrival Times: The interarrival time gaps between the vehicles have been found to follow a shifted exponential distribution (Sec. 2.3) in which the probability P of a gap being less than t seconds is given by;

$$P\,(h < t) = \left[1 - e^{-\frac{(t-1.0)}{(T/V-1.0)}}\right] \qquad (5.26)$$

or $\quad t = 1.0 - (T/V - 1.0)\,\mathrm{Log}_e\,(1 - P) \qquad (5.27)$

where (1-P) is a random fraction whose value lies within (0-1) range. By Monte Carlo method if a uniformly

distributed random number of (0-1) range is generated and substituted for (1-P) in Eq. 5.27, the interarrival time gaps can be computed (Von Neumann, 1951; Gerlough, 1964). These time gaps follow the shifted exponential distribution. From the sequence of arrival gap lengths t_i , the arrival time of vehicles $AT_i = AT_{i-1} + t_i$ were computed. Separate sequence of arrival times were computed for the two lanes.

Category of Arriving Vehicles: Vehicles have been classified into six different categories. When a new vehicle enters the section from either direction, it is necessary to know its category so that other characteristics like free speed etc. may be generated accordingly. It is possible to specify for any category of vehicle a subrange corresponding to composition in the overall range of (0-1) for $F(x)$. Any arbitrary allocation of the subranges is possible, for example, the various subranges corresponding to a composition are shown in Table 5.2. The subranges for any desired composition are stored. A uniformly distributed random number (r) is generated and its value compared with various subranges. When r lies in a particular subrange, the arriving vehicle belongs to the category of that subrange.

TABLE 5.2 COMPOSITION AND CHARACTERISTICS OF MIXED VEHICULAR TRAFFIC

K	Category of Vehicle	Assumed Traffic Composition in Percent	Subrange	Mean Free Speed MVF(K)	Standard Deviation of Free Speed SDVF(K)
1	Cars	20	$0 \leq r \leq 0.20$	49.95	8.05
2	Trucks	15	$0.20 \leq r \leq 0.35$	44.30	6.80
3	Tongas	5	$0.35 \leq r \leq 0.40$	12.80	1.50
4	Bullock Carts	10	$0.40 \leq r \leq 0.50$	5.20	1.10
5	Scooters	10	$0.50 \leq r \leq 0.60$	39.80	7.60
6	Bicycles	40	$0.60 \leq r \leq 1.0$	11.90	1.80

Free Speed of Vehicles: The free speeds of a particular category of vehicle follow a censored normal distribution with the range ($\bar{x}$ - 3 sd) to ($\bar{x}$ + 3 sd). The central limit theorem can be used to derive a generator (RN) for normally distributed random variables by taking the sum of twelve uniform variables (Tocher, 1963), i.e., $RN = (\sum_{i=1}^{12} r_i - 6.0)$. Knowing the normally distributed random variable RN , the free speed of vehicle FS of category K was determined as follows:

$$FS = MVF(K) + RN\ (SDVF(K)) \qquad (5.28)$$

where MVF(K) and SDVF(K) are the mean and standard deviation of free speeds for category K.

Uniformly distributed random numbers are needed for generating input data. In all, sixteen sets of random numbers were generated using the same subroutine by changing the initial value n_o, viz.,

(i) two sets for arrival gap distribution of either direction;

(ii) two sets for determining the category of arriving vehicles, i.e., one for each lane; and

(iii) twelve sets for free speed distribution of different categories of vehicles (six for each lane).

5.6 Measures of Effectiveness

Goals and objectives of a study should be expressed in explicit terms so that definite measures can be used in evaluating the system behaviour. Commonly such measures are termed as criteria, figures of merit or measures of effectiveness (Wohl and Martin, 1967). These have three fundamental characteristics, viz., (i) they should be quantitative, (ii) they should measure the effectiveness of the entire system; and (iii) they should be efficient statistically, i.e., have a small variance.

Quantitative measures of effectiveness that were used to validate the simulation model and to estimate the flow characteristics are: (i) travel time and hence operating speed for each category of vehicle in the mixed flow; (ii) proportion of delayed vehicles and delay times of different categories; and (iii) density of the section.

5.7 Initial Conditions

A process may be steady or transient. A transient process has to be studied from its actual initial conditions for the desired time period. But for a steady state process, it is necessary to ensure that the characteristics of the process have reached a steady state. Simulation of the steady state process is carried out for a sufficiently long period in order to estimate the characteristics of the process. It is necessary to select the starting conditions that make the transient period short. Starting conditions can be set by any of the following methods (Mize and Cox, 1968) :

(i) Choose starting conditions that approximate steady state conditions and insert explicity these starting conditions in the model;

(ii) Use a long period simulation so that the data from the transient period is insignificant relative to the steady state data;

(iii) Introduce an initial nonrecording period to get the process into steady state conditions. This is done by running the process till steady state conditions are achieved, then clearing all statistical accumulations (but leaving the state of system as it is) and continuing the run for a sufficient period.

Simulation runs are made for two cases, viz.,
(i) for validation of the formulated mathematical model; and (iii) for estimation of the characteristics of the process under steady state for varying volume levels and traffic composition.

For mixed vehicular traffic at different compositions, the steady state conditions are not known and hence it is not possible to adopt the first method of starting with known steady state conditions. Second method involves errors in estimation of parameters as transient effects may become insignificant only for extremely long times. Hence the following procedure based on the third method was adopted in the study.

Simulation is started with an empty system. The vehicles start arriving from both directions. First few vehicles are not significantly affected by the traffic stream and move at their free speeds. Constraints start

building up when vehicles arriving from either direction occupy the entire length of the roadway section. Steady state conditions are said to exist when rate of arrival equals the rate of departure. These conditions are likely to occur only when some vehicles have passed through the entire section. The initial time period is affected by the length of the roadway section. Trial runs were made at various volume levels to obtain the steady state conditions. It was observed that time period for initialisation is also governed by the volume level; being less for high volume of traffic. This appears to be logical as with higher volume levels, arrival gaps are less and vehicles being closely spaced, approach the steady state conditions soon. Trial runs showed that for a 2.25 km. long section, the maximum time period for initialisation was around 15 minutes for a traffic volume of 50 vehicles per hour per lane, whereas it was only 5 minutes for volume level of 700 and more. Based on the trial runs, initial periods selected for simulation experiments are given in Table 5.3. For some volume levels and compositions, jamming occurs and hence there are no steady state conditions. In this case also, the time periods for initialisation given in Table 5.3 were adopted.

TABLE 5.3 INITIAL TIME PERIODS FOR OBTAINING STEADY STATE CONDITIONS

Traffic Volume VPH in Either Direction	Initial Time Period in Seconds	
	1.0 km. Section	2.25 km. Section
50 - 100	600	900
150 - 300	500	600
350 - 600	300	400
650 - 1000	200	300

5.8 Stopping of Simulation Runs

Simulation experiments on a model can be conducted either at a particular point in time or over extended periods of time. In the former case, simulation is said to be static in which sampling takes place across the ensemble. In the latter case, the simulation is said to be dynamic,i.e., time series simulation in which sampling takes place over time (Naylor et al , 1971). A static simulation is achieved by replicating a given simulation run by changing the sequences of pseudorandom numbers used to generate stochastic variables. Dynamic simulation results by simply extending the length of a run over time. The main problem is how long the sample record must be (Number of replications in static simulation

and time period for dynamic simulation) in order to achieve a given level of statistical precision. A number of stopping rules are generally adopted and they are available from references (Bechhofer, 1954; Winer, 1962; Gilman, 1968).

The sample size may be estimated by (i) finite sampling techniques where the sample size is initially determined for a given level of accuracy, or (ii) sequential sampling techniques where the sample is tested at frequent intervals to determine the error of estimate and to stop sampling when the errors are within appropriate limits. A sequential procedure was adopted in this study.

The main flow characteristics in this study are the delay time and operating speed of vehicles. At the end of initialisation, the system is in a steady state and system characteristics reach their steady state values. They will remain unchanged over further periods of simulation. The output characteristics are evaluated only after the initial time period. The average value of the characteristics were calculated for successive intervals of 5 minute duration. Operating speed of cars was used as the dominant characteristic. Simulation was stopped when the average operating speed of cars for two consecutive intervals differ by not more than 0.1 kmph. Simulation time was a function of traffic volume and it varied from around 30 minutes for high volume levels

of around 800 to more than an hour for volume levels of the order of 100.

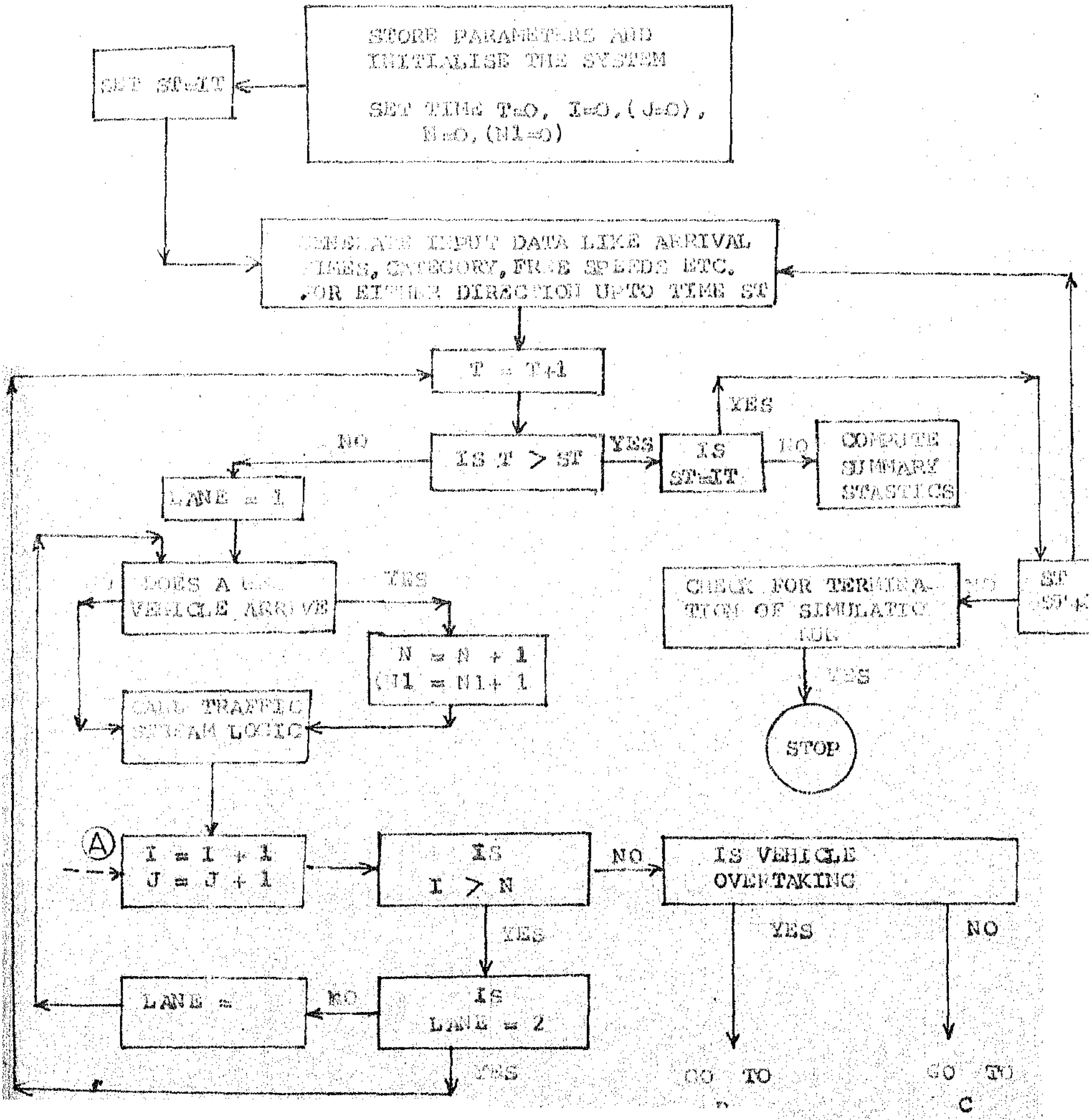
5.9 Formulation of Computer Programme

5.9.1 General

Flow charts for computer simulation of mixed vehicular traffic are shown in Figs. 5.5 and 5.6. The computer programme may be written in either a general purpose language like FORTRAN or a special simulation language like GPSS (Schriber, 1970), SIMSCRIPT (Kiviat, 1968), DYNAMO (Pugh, 1963) or SIMULATE (Hold, 1967). The special simulation languages have been written to facilitate the programming of certain types of systems. For example SIMULATE was designed for simulating large size economic systems whereas GPSS/360 and SIMSRIPT are particularly suited for queuing problems. The principal advantage of these languages is that when found suitable, they require less programming effort and time than for FORTRAN programming.

5.9.2 Computer Programming

Mixed vehicular traffic flow is quite complex and simulation languages available were not considered to be suitable for simulation. Furthermore, memory requirement was also a constraint. Hence FORTRAN language was used for



STORE PARAMETERS AND INITIALISE THE SYSTEM
SET TIME T=O, I=O, (J=O), N=O, (N1=O)
SET ST=IT
GENERATE INPUT DATA LIKE ARRIVAL TIMES, CATEGORY, FREE SPEEDS ETC. FOR EITHER DIRECTION UPTO TIME ST
T = T+1
IS T > ST
NO
YES
IS ST=IT
NO
COMPUTE SUMMARY STASTICS
YES
LANE = 1
DOES A VEHICLE ARRIVE
YES
N = N + 1
(N1 = N1+ 1
CALL TRAFFIC STREAM LOGIC
CHECK FOR TERMINATION OF SIMULATION RUN
NO
YES
STOP
A
I = I + 1
J = J + 1
IS I > N
NO
IS VEHICLE OVERTAKING
YES
NO
YES
LANE =
NO
IS LANE = 2
YES
GO TO
GO TO
C

5.5 CONTD...

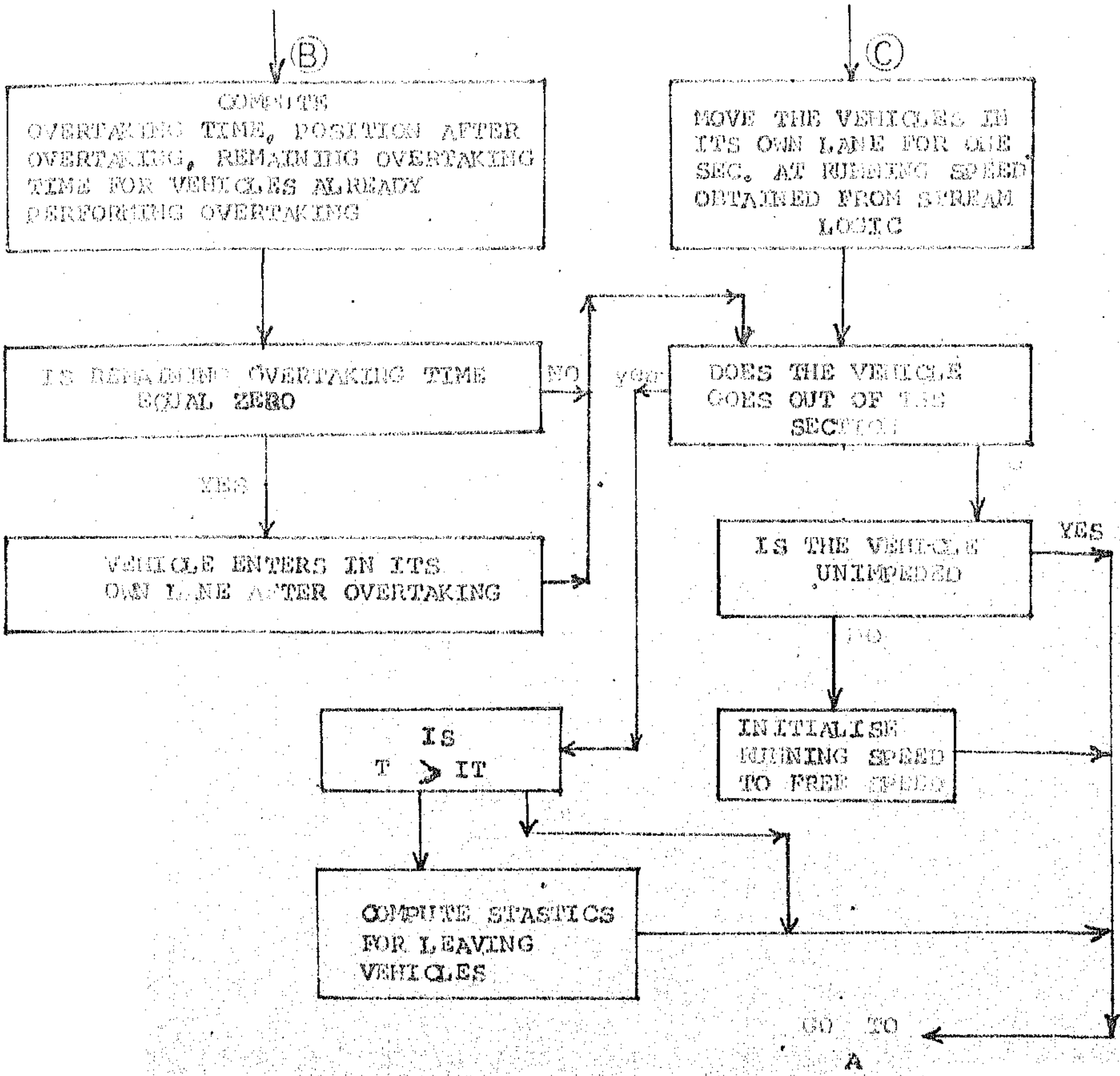


FIG. 5.5 FLOW CHART FOR SIMULATION PROGRAMME

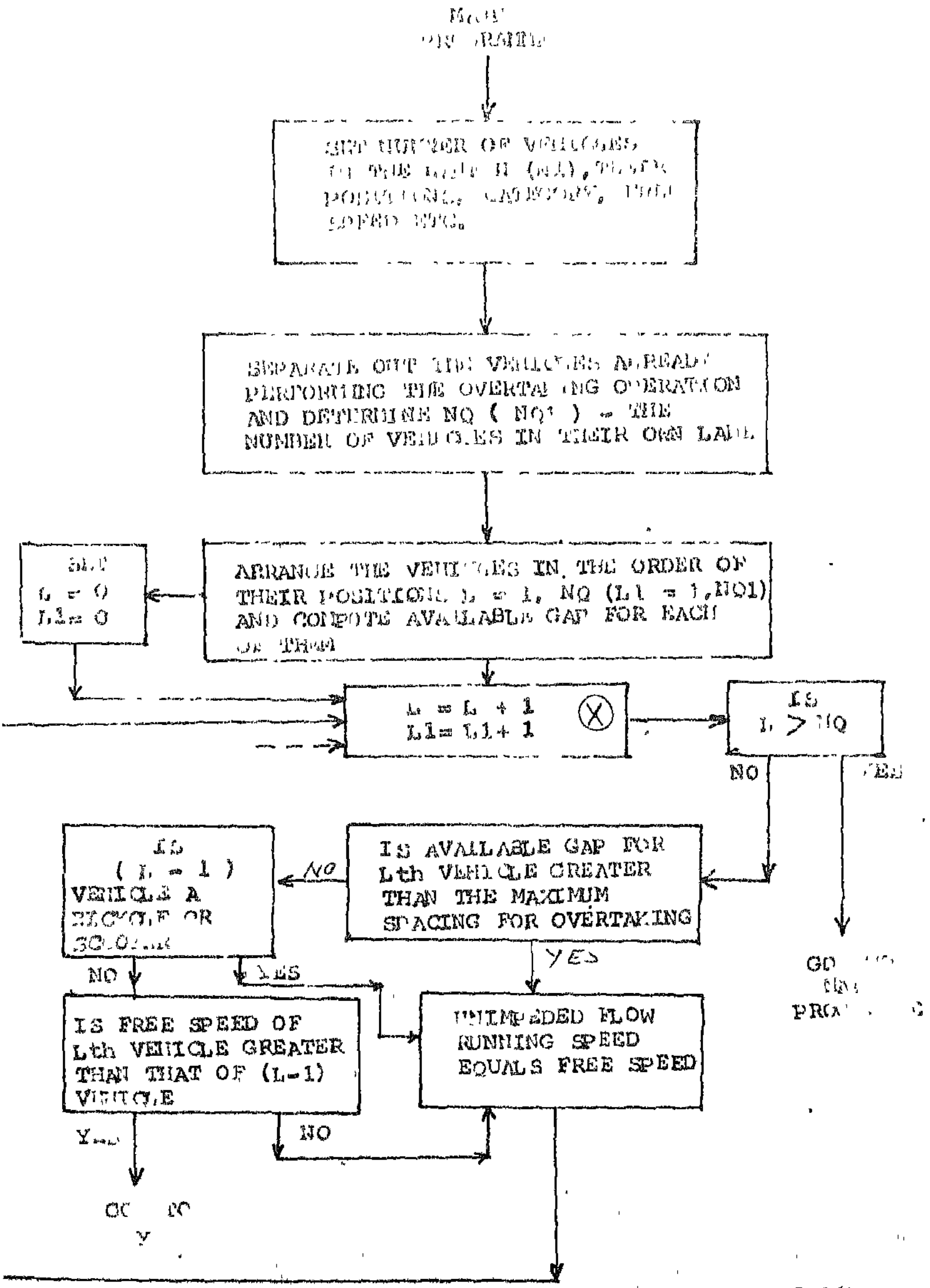

MAIN PROGRAM
SET NUMBER OF VEHICLES IN THE LANE II (NL), THEIR POSITIONS, CATEGORY, THEIR SPEED ETC.
SEPARATE OUT THE VEHICLES ALREADY PERFORMING THE OVERTAKING OPERATION AND DETERMINE NQ (NQ1) - THE NUMBER OF VEHICLES IN THEIR OWN LANE
ARRANGE THE VEHICLES IN THE ORDER OF THEIR POSITIONS L = 1, NQ (L1 = 1,NQ1) AND COMPUTE AVAILABLE GAP FOR EACH OF THEM
SET L = 0 L1= 0
L = L + 1 L1= L1+ 1
X
IS L > NQ
NO
YES
IS AVAILABLE GAP FOR Lth VEHICLE GREATER THAN THE MAXIMUM SPACING FOR OVERTAKING
NO
YES
IS (L - 1) VEHICLE A BICYCLE OR SCOOTER
NO
YES
IS FREE SPEED OF Lth VEHICLE GREATER THAN THAT OF (L-1) VEHICLE
UNIMPEDED FLOW RUNNING SPEED EQUALS FREE SPEED
YES
NO

FIG. 5.6 CONTD.....

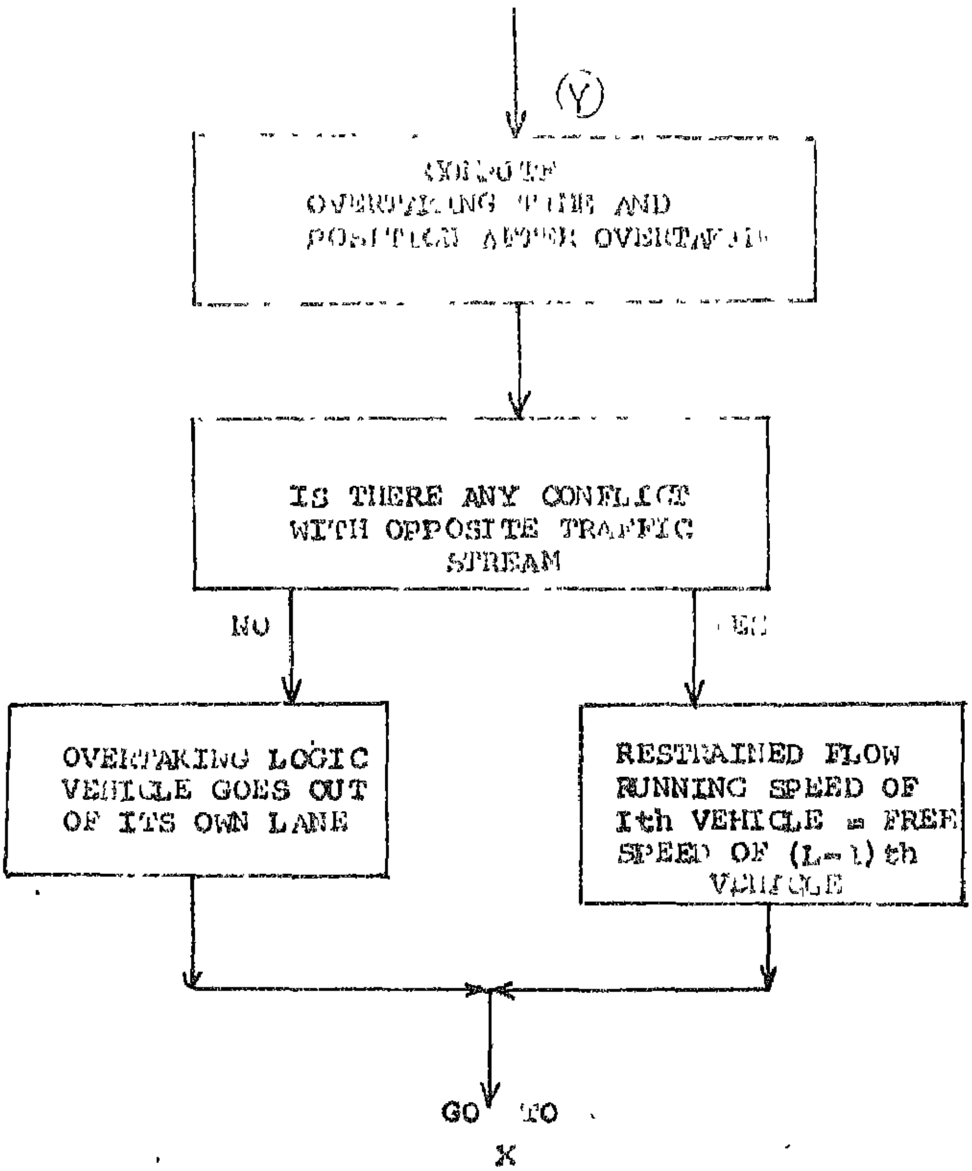


FIG. 5.6 FLOW CHART FOR TRAFFIC STREAM LOGIC

computer programming. Simulation consists essentially of the following:

(i) The volume level and composition of traffic is read;

(ii) Initially the system is assumed to be empty;

(iii) Vehicular arrival times and other related characteristics are generated and stored for every 5 minute intervals;

(iv) Scanning is done at 1 second intervals. The vehicles are arranged in the order of their positions, their stream logic is determined and they are moved through appropriate distances;

(v) The process is run for the initial time period in order that a steady state may be reached;

(vi) The process is simulated for 5 minute intervals, at the end of which the output is processed and printed; and

(vii) Simulation is stopped when the parameter estimates are within specified error limits.

The programme consists of six subroutines and a main programme to link them. They include:

(i) SUB1 and SUB2 - to decide the stream logic for vehicles in the stream,

(ii) RA and NA - to generate pseudorandom numbers of uniform and normal distributions respectively;

(iii) MEETU - to separate the output variables like average delay, operating speed, free speed, density etc. for six different categories of vehicles; and

(iv) AVSD - to compute the mean and standard deviation of various output variables for each category of vehicle.

The programme involves large number of variables and the maximum dimension of any variable representing vehicular characteristics can be 100 only. In simulation, a larger number of vehicles need to be processed. A chain was set up to avoid memory overflow. As soon as the sequence number of arriving vehicles reached 100 , the departing vehicles were taken out of the system and sequence number of vehicles alongwith their characteristics were changed. New vehicles were generated with the sequence number not exceeding 100 at any time. By this chain system, the dimension was restricted to 100 without affecting the number of vehicles to be processed.

5.10 Validation of Model

5.10.1 General

Formulation of a mathematical model involves the definition of a structure and a set of parameters. Validation involves the verification that the structure is correct and the parameter estimates are reasonable. Validation may also

involve parameter estimation. Generally the validation of the model is based on available information and an agreement between the throughput or output of the model with earlier observations. The better the agreement, better is the validity of the model. The model can be used to simulate future conditions only after it has been validated and found to be a reasonable representation of the actual system under study (Naylor and Finger, 1967).

5.10.2 Parameter Estimation and Validation

A simulation model and the constituent equations have been described earlier in Secs. 5.2 and 5.3. It involves assumptions of specific relations for minimum spacing S(I), and maximum spacing for overtaking vehicles SD(I) in terms of parameters L_o and T_o (Eqs. 5.8 and 5.9), and the reaction time of looking for gap T_R , which is also a parameter. Initially the values of L_o , T_o and T_R were assumed on the basis of comparable I.R.C. (1950) Specifications for overtaking sight distances. Validation was not satisfactory and so it was proposed to treat them as parameters and estimate their values by simulation.

From records of field studies, data from six periods of one hour each with different compositions and volume levels, were selected for parameter estimation and validation.

For each hour, arrival time, category and free speed of vehicles were generated on the basis of fitted distributions. Using estimated values of parameters, the process was simulated. There are several measures of effectiveness. It was proposed to use for validation the frequency distribution of delay times for cars and trucks which are most affected by interaction. Simulation was done until these parameters could be estimated within the specified level of error. They were compared with field data for goodness of fit by χ^2- test. When the fit is not satisfactory, the parameter values L_O , T_O and T_R were modified till the simulated results were comparable to field data. In this study the model was validated after four trials. The fitted parameters were as follows:

(i) Minimum spacing for vehicles;

$$S(I) = 0.2\ V(I) + L_O$$

where L_O = 6.5 metres for heavy motor vehicles, bullock carts and tongas

= 5.5 metres for passenger cars; and

= 4.0 metres for scooters and bicycles

(ii) Headway gap (T_O) of 2 seconds, i.e., SD(I)=0.56 V(I).

Overtaking is likely to take place only when the available headway equals or is less than 2 seconds.

(iii) Reaction time (T_R) of looking for gap in the opposite stream equals 2 seconds.

For the fitted model, validation also indicated a general agreement with frequency distribution of operationg speeds of different categories of vehicles and the proportion of delayed vehicles. The model is considered to be a satisfactory representation of the system and so it can be used for further computer simulation experiments with mixed vehicular traffic.

6. SIMULATION ANALYSIS OF MIXED VEHICULAR TRAFFIC

6.1 Design of Simulation Experiments

6.1.1 Introduction

The behaviour of a system is affected by a number of factors. Simulation essentially deals with the evaluation of system response to changes in the factors affecting it. The aim of any experimental investigation might be to explore and describe the response surface over regions of interest in the factor space. This needs observations of the response at various factor levels. Some of the criteria for good experimental design are as follows (Cochran and Cox, 1957):

(i) The model and its underlying assumptions should be appropriate;

(ii) The analysis resulting from design should provide unambiguous information on the primary objective of the experiment;

(iii) The design should provide maximum information with respect to the major objectives and adequate information with respect to all objectives of the experiment ; and

(iv) The design must be feasible within the working conditions that exist for the experimenter.

There are a set of experimental designs that provide not only economy in the required number of experimental trials, but also additional qualities like minimum variance estimates, desirable confounding patterns and ease of computation. The common methods of experimental design are full factorial, fractional factorial, rotatable, and response surface designs. (Davies, 1960 ; Bonini, 1963 ; Herzberg and Cox , 1969).

6.1.2 Sampling for Parameters

Interaction between the vehicles affects the basic flow characteristics of mixed traffic at various volume levels. Important parameters include the following:

(i) volume of the mixed traffic in either direction;

(ii) its composition, i.e., proportion of the different categories of vehicles in the mix ; and

(iii) the free speed distributions of the different categories of vehicles.

In the present study, the free speed distribution parameters of different categories of vehicles were kept constant. Only traffic composition and volume levels were selected for exploring the response surface. Traffic volumes and compositions were also kept same in either direction.

The operating speed of the slow moving vehicles does not depend generally on the traffic composition and volume levels as these vehicles can move at their free speeds irrespective of other traffic factors. On the other hand, the operating speed of motor vehicles is very much dependent upon the traffic factors and is below their free speeds due to restrained operations. Operating speed defines the level of service for any vehicle and interaction affects the level of service for cars which have the maximum speed among all the six categories. The experiment was designed for various combinations of different vehicles at varying proportions. For each combination, the response was estimated for increasing volume levels till at a particular volume, capacity is exceeded, resulting in platooning of vehicles.

6.1.3 Experimental Design for Homogeneous Traffic

Simulation experiments were designed for homogeneous traffic consisting of either passenger cars or trucks only. These two vehicles move faster amongst all the vehicles and their operating speeds are likely to be affected under different sets of traffic conditions. The free speeds of vehicles of each category are probabilistic, and there is considerable interaction between vehicles of the same type

even in homogeneous flow. The volume level for each simulation run was increased till jamming occurs. The volume levels of passenger cars and trucks used for simulation are shown in Table 6.1.

6.1.4 Experimental Design for Mixed Traffic

Experimental design for mixed traffic requires the selection of different combinations of vehicles. Each of the combinations is to be experimented at various volume levels. Factorial designs require a very large number of design points to map the entire response surface. This was not possible due to constraints on computer time. Based on different traffic compositions, a few selected design points were used to estimate the interactions between different combinations of vehicles.

The interaction between passenger cars and any other category of vehicle was evaluated for all the five possible combinations, viz., each case included passenger cars and one of the remaining five categories with different proportions of vehicles in the mix. The proportions selected for design were 30 and 50 percent of trucks ; 25 and 50 percent of scooters; 25 , 50 and 75 percent of bicycles; 10 , 25 and 50 percent of bullock carts; and 10 and 25 percent of tongas. The volume level for each set was increased till it exceeded

TABLE 6.1 FACTOR COMBINATIONS FOR SIMULATION OF MIXED VEHICULAR TRAFFIC

Combinations	Traffic Composition in Percent		Volume Level(V) in Either Direction-VPH
Homogeneous	Cars		50,100,150,200,300,400,500,600,650,700,800
	Trucks		50,100,150,200,300,400,500,600
Two Vehicle Combinations	Cars	Trucks	
	70.0	30.0	50,100,200,300,400,500,600
	50.0	50.0	50,100,200,300,400,500
	Cars	Tongas	
	90.0	10.0	100,200,300,400,500
	75.0	25.0	100,200,300,400,450
	Cars	Bullock Carts	
	90.0	10.0	100,200,300,350
	75.0	25.0	100,200,300
	50.0	50.0	100,200
	Cars	Scooters	
	75.0	25.0	100,200,250,300,400,600,800,850
	50.0	50.0	100,200,300,400,600,800,900,1000
	Cars	Bicycles	
	75.0	25.0	100,200,300,400,600,800
	50.0	50.0	100,200,300,400,600,800,900
	25.0	75.0	100,200,300,400,600,800,900,1000

Contd....

TABLE 6.1 CONTD....

Combinations	Traffic Composition in Percent			Volume Level(V) in Either Direction-VPH
Three Vehicle Combinations	Cars	Trucks	Tongas	
	63.0	27.0	10.0	100,200,300
	45.0	45.0	10.0	100,200,300
	52.5	22.5	25.0	100,200,300
	37.5	37.5	25.0	100,200,300
	Cars	Trucks	Bullock Carts	
	63.0	27.0	10.0	100,200,300
	45.0	45.0	10.0	100,200
	52.5	22.5	25.0	100,200
	37.5	37.5	25.0	100,200
	35.0	15.0	50.0	100
	Cars	Trucks	Scooters	
	52.5	22.5	25.0	100,200,300,400,600
	37.5	37.5	25.0	100,200,300,400,600
	35.0	15.0	50.0	100,200,300,400,600,800
	25.0	25.0	50.0	100,200,300,400,600,800
	Cars	Trucks	Bicycles	
	52.5	22.5	25.0	100,200,300,400,600,800
	37.5	37.5	25.0	100,200,300,400,600
	35.0	15.0	50.0	100,200,300,400,600,800
	25.0	25.0	50.0	100,200,300,400,600
	17.5	7.5	75.0	100,200,300,400,600,800
	12.5	12.5	75.0	100,200,300,400,600,800
Six Vehicle Combinations	Cars=17.5, Trucks=7.5, Tongas=6.0, Bullock Carts=4.0, Scooters=15.0, Bicycles=50.0			100,200,300,400
	Cars=15.0, Trucks=15.0, Tongas=6.0, Bullock Carts=4.0, Scooters=10.0, Bicycles=50.0			100,200,300,400

capacity. The various sample points are also given in Table 6.1.

The interaction between three categories of vehicles was evaluated for four combinations. These include cars, trucks and one of the remaining four categories of vehicles, viz., bullock carts, tongas, scooters and bicycles. Mixed traffic flow affects the characteristics of fast vehicles like cars and trucks and these two vehicles also interact with each other to a considerable extent. The car - truck ratio was maintained as 70/30 and 50/50 as in the case of two vehicle interaction studies. The combined effect due to the three vehicles was evaluated for different proportions of the third vehicle in the mix for the two car - truck ratios in each of the four combinations. The proportions of the third vehicle in the total volume were 25 and 50 percent of scooters ; 25 , 50 and 75 percent of bicycles ; and 10 and 25 percent for both tongas and bullock carts. They were simulated at various volume levels given in Table 6.1.

The interaction between four and five categories of vehicles in the mix were not investigated. Yet the interaction between all the six categories of vehicles in the traffic mix was estimated at the two observed peak traffic compositions at different volume levels (Table 6.1).

In all about 180 experimental runs for different compositions and volume levels of various combinations of vehicles were made requiring more than 100 hours of processing time on IBM 7044/1401 system at I.I.T. , Kanpur.

6.2 Simulation Characteristics

Simulation for each set of data was started on an empty system till steady state conditions were achieved at the end of the initial time period. Simulation was continued further and output variables were determined at the end of every 5 minute interval till the parameter estimates were within specified error limits. The specific characteristics obtained from simulation runs include:

(i) Proportions of each category of vehicles delayed and their delay times;

(ii) Operating speeds of different vehicles (VEF) and also of different categories, (AVVE);

(iii) Density of the section or the number of vehicles of each category per unit length ; and

(iv) Rate of departure of each category of vehicles from the section.

6.3 Homogeneous Traffic Flow

In homogeneous traffic , the interaction is between vehicles of the same category. Simulation of homogeneous traffic consisting of only cars or trucks was performed at various volume levels.

6.3.1 Homogeneous Car Traffic

Simulation results of homogeneous car traffic are shown in Figs. 6.1 to 6.5. At low volume levels, say upto 150 vehicles per hour (VPH) for either direction, the available headways are more and very few manoeuvring operations are restrained. This results in very samll proportion of the vehicles being delayed (Fig. 6.1) ; the average delay for all the vehicles at low volumes is very small(Fig.6.2)there are very few vehicles in the section, i.e., density is low (Fig. 6.3) ; and operating speed of cars is very near their free speeds (Fig. 6.4). As volume increases, the available spacings get reduced and in turn a large number of overtaking vehicles are restrained. The increased volume thus increases the delay and density thereby considerably reducing the operating speed. Simulation results (Figs. 6.1 to 6.5) indicate the following four distinct characteristics:

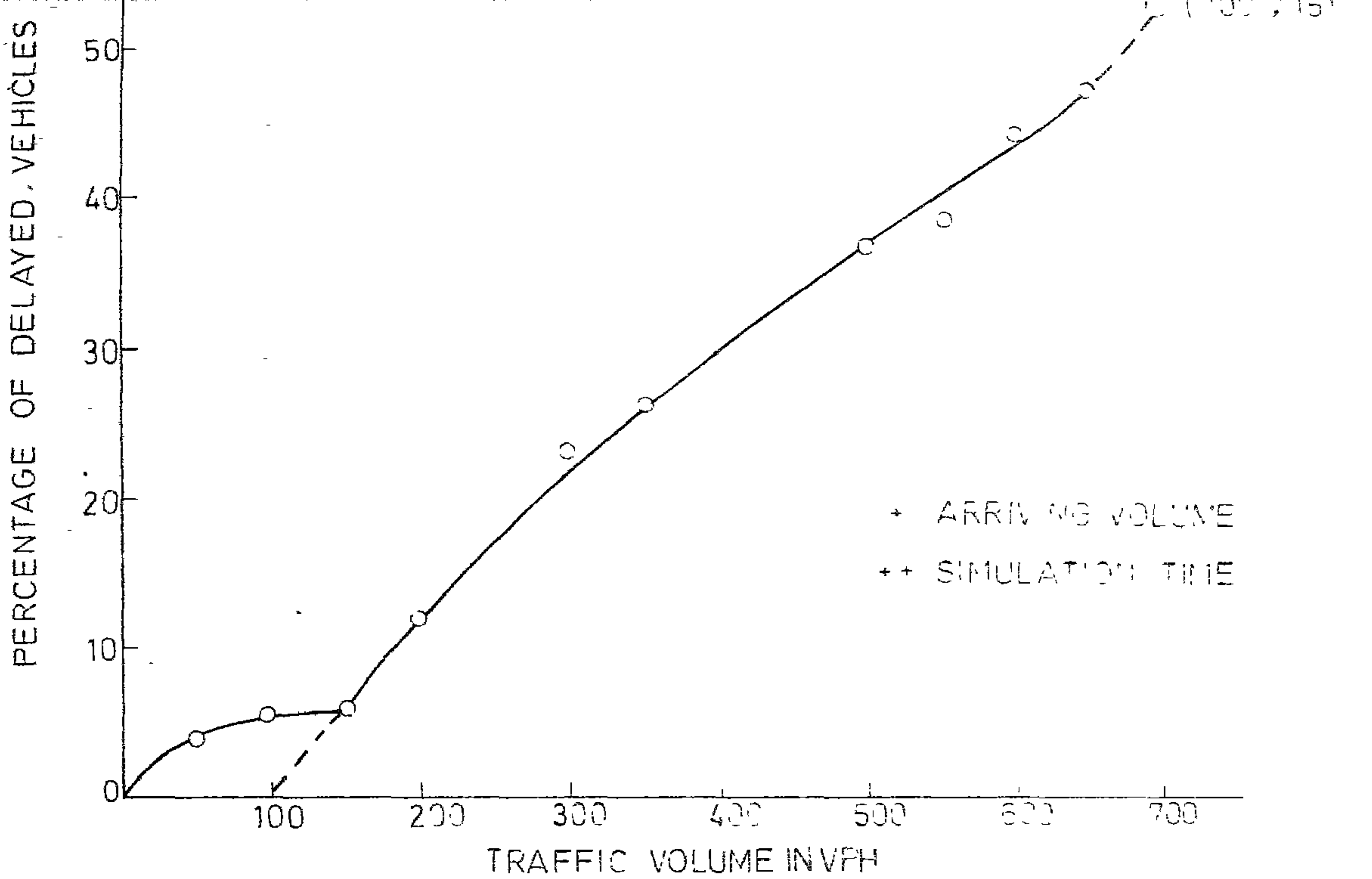
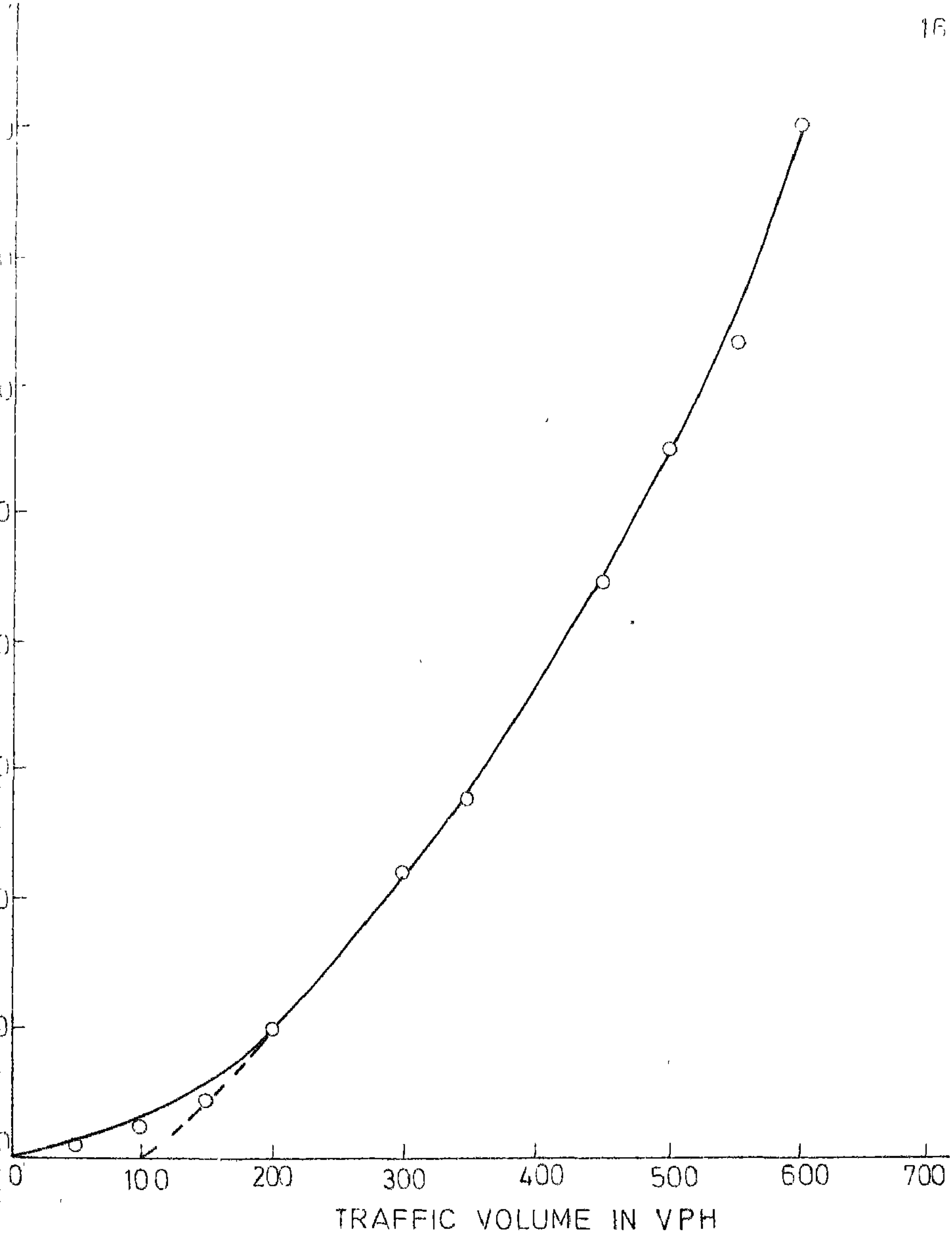
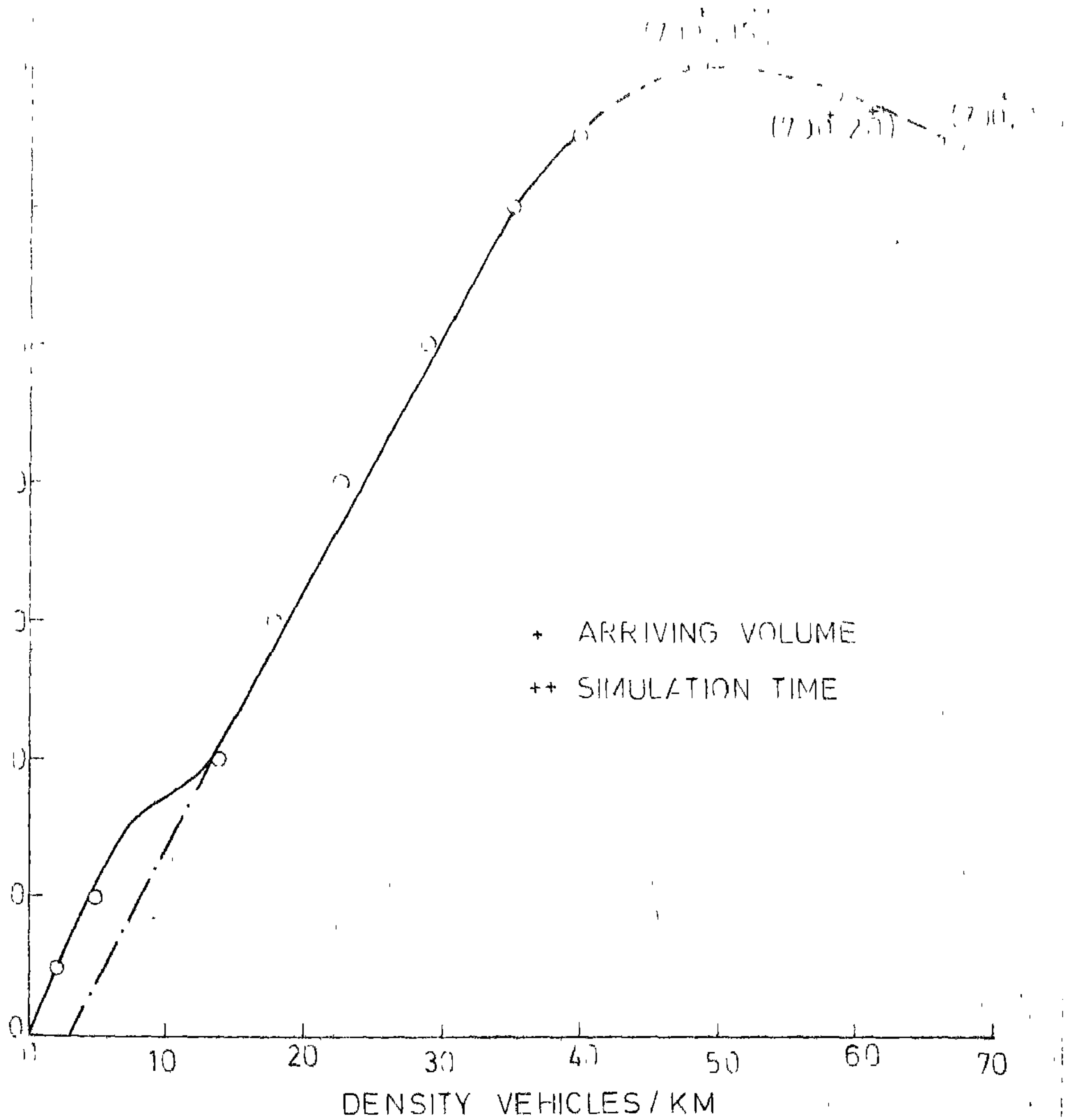


FIG.6·1 PROPORTION OF DELAYED VEHICLES IN HOMOGENEOUS CAR TRAFFIC



G.6.2 AVERAGE DELAY TIME OF CARS AT VARIOUS VOLUME LEVELS



63 VOLUME DENSITY RELATIONSHIP FOR HOMOGENEOUS CAR TRAFFIC

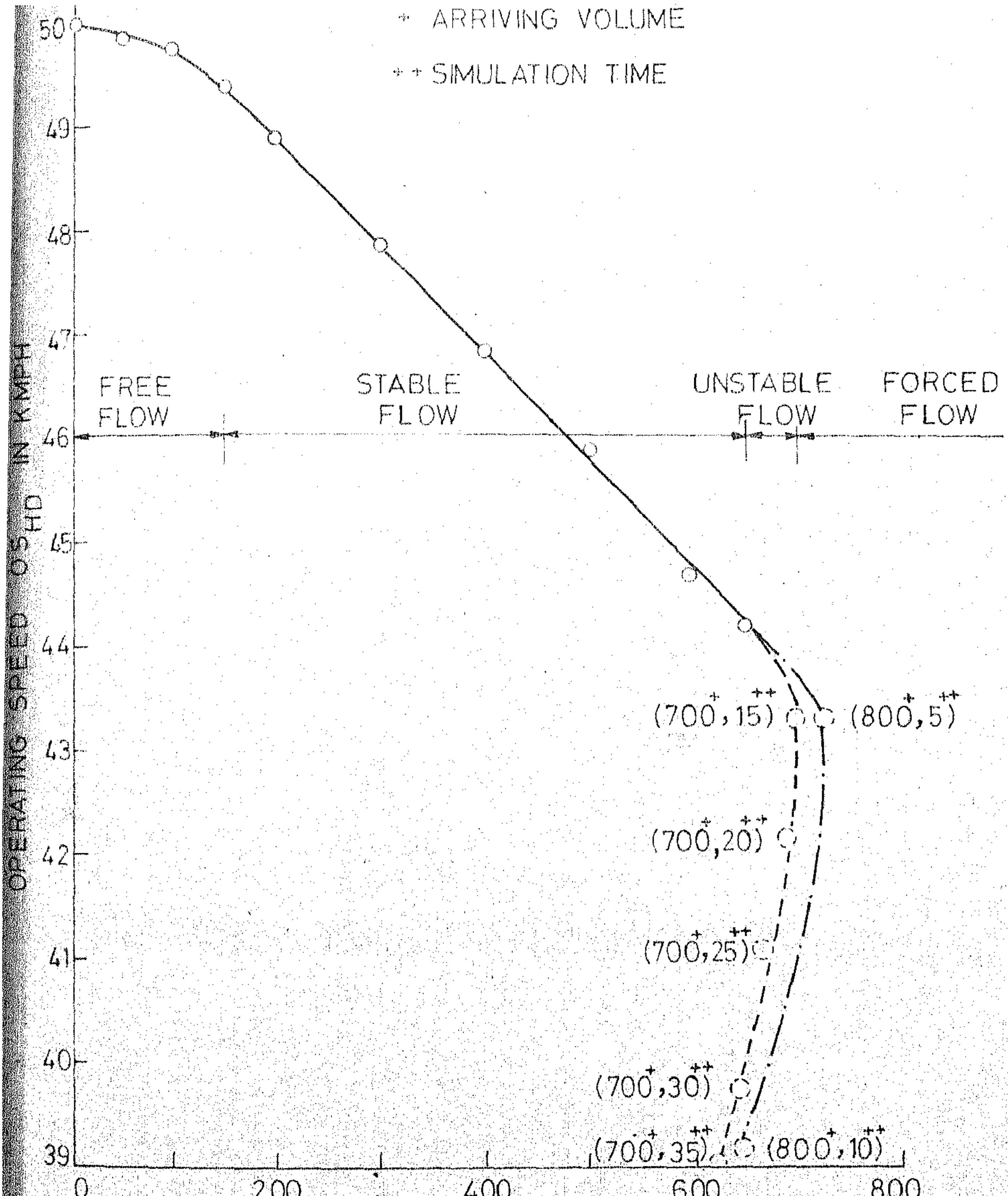
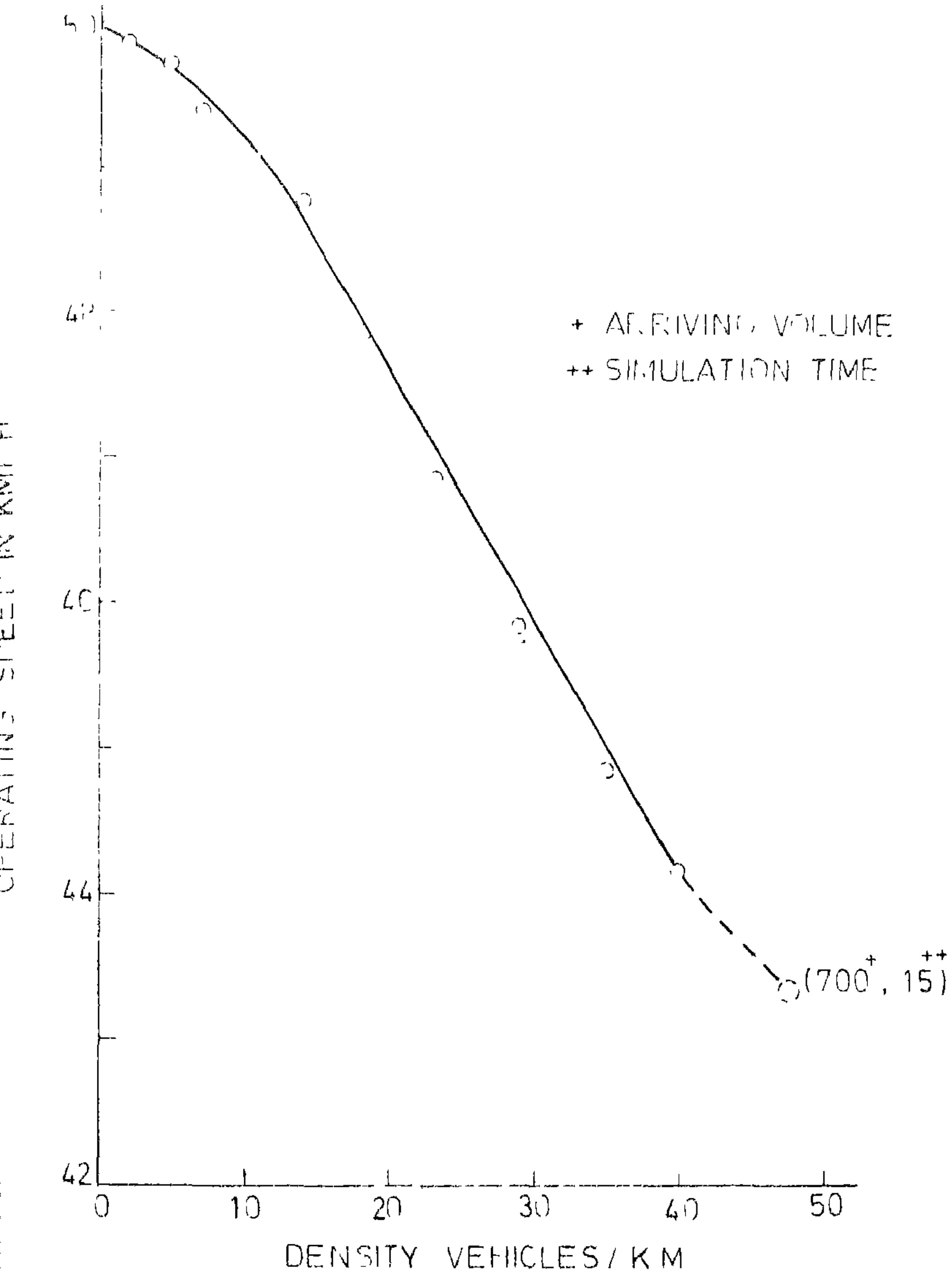


FIG.6.4 SPEED VOLUME RELATIONSHIP FOR HOMOGENEOUS CAR TRAFFIC



FIG·6·5 SPEED DENSITY RELATIONSHIP FOR HOMOGENEOUS CAR TRAFFIC

(i) Upto a volume level of 150 VPH in either direction, the rates of increase of delayed vehicles and average delay time are very small. Density also builds up slowly and operating speed - volume relationship is nonlinear with operating speed very near the free speed.

(ii) Beyond volume level of 150 VPH , there is a sudden change in the proportion of delayed vehicles,density increases sharply, delay times increase, and the operating speed also decreases at a rapid rate. Between volume levels of 150 to 600 , there is a regularity in the relationships between the flow characteristics. Operating speed and the density change linearly whereas the proportion of the delayed vehicles and the delay times increase non-linearly with volume.

(iii) Between volume levels of 600 and 650 , the proportion of the delayed vehicles, delay times and the density increase rapidly. The operating speed also decreases at a faster rate.

(iv) At a volume level of 700 VPH , traffic flow is always in the transient state and steady state is never reached. The characteristics of flow change with time and ? are indicated by marked open circles in Figs. 6.1 , 6.3 and 6.4. These indicate that density of vehicles is building up in the road because of jamming with a consequent

increase in delay time and decrease in operating speed and hence the volume.

Based on simulation results, the following mathematical relationships have been established by regression analysis.

The proportion of the delayed vehicles is given by

$$P_d = 0.135 (V-100) + 0.0001 (V-100)^2$$
$$\text{for } 150 \leqslant V \leqslant 600 \qquad (6.1)$$

where P_d = percentage of delayed vehicles and V = traffic volume in either direction (VPH).

The average delay per vehicle increases with volume level and is represented as

$$\bar{d} = 0.0045 V + 0.00002 (V-100)^2$$
$$\text{for } 150 \leqslant V \leqslant 600 \qquad (6.2)$$

where $\bar{d}$ = average delay in seconds.

The volume density relationship shown in Fig. 6.3 indicates an abrupt increase in density between volume levels of 150 to 200 , though the rate of increase in density upto volume level of 150 is same as that between 200 to 600 VPH. The relationships can be represented as:

$$D_t = 0.054 V \quad \text{for} \quad V \leqslant 150 \qquad (6.3)$$

and $D_t = 13 + 0.054 (V-200)$

$$= 2.2 + 0.054 V \quad \text{for} \quad 200 \leq V \leq 600 \qquad (6.4)$$

where D_t is the density in terms of vehicles per km. length of road section.

Beyond volume level of 600 , the density increases rapidly and at 700 VPH , the density continues to increase with time while the outgoing volume rate decreases indicating jamming. Jamming seems to occur when density exceeds around 40 vehicles per lane per km. of roadway section.

The speed volume relationship (Fig. 6.4) indicates that upto a volume level of 150 VPH , the operating speed decreases nonlinearly and is very near the free speed. It can be represented as

$$OS_{HT} = 50 - 0.0014 V - 0.000019 V^2$$

$$\text{for } V \leq 150 \qquad (6.5)$$

where OS_{HT} = operating speed (kmph) of homogeneous car traffic having mean free speed of 50 kmph.

Between traffic volume of 150 to 650 , the relationship can be given by

$$OS_{HT} = 49.36 - 0.0101 (V - 150)$$

$$= 50.875 - 0.0101 V \qquad (6.6)$$

$$\text{for } 150 \leq V \leq 650$$

For a volume of 700 VPH, the operating speed decreases to 43.28 kmph after 15 minutes of simulation and to 39 kmph after 30 minutes. The outgoing traffic rate is around 650 VPH at the end of 30 minutes. With an input volume of 800 VPH the jamming starts in just 5 minutes of simulation and operating speed decreases to 39 kmph in 10 minutes. It is generally seen that jamming starts when operating speed of cars is around 43.30 kmph.

The speed density relationship can be derived from Eqs. 6.3 to 6.6 as follows:

$$OS_{HT} = 50 - 0.0014\, V \left(\frac{D_t}{0.054} \right) - 0.0000\,19 \left(\frac{D_t}{0.054} \right)^2 \qquad (6.7)$$

$$\text{for } D_t \leqslant 8$$

and

$$OS_{HT} = 50.875 - 0.0101 \left(\frac{D_t - 2.2}{0.054} \right) \qquad (6.8)$$

$$14 \leqslant D_t \leqslant 40$$

Fig 6.5 shows that the speed density relationship is nonlinear at lower density (less than 8)and is linear for densities between 14 to 40 vehicles per lane per km. This agrees with Eqs. 6.7 and 6.8.

Capacity and Level of Service: The concept of level of service can be defined best in terms of measures familiar to drivers. H.R.B. (1965) specifies the level of service

by: (i) operating speed; and (ii) ratio of service volume to capacity.

In order that a highway provides an acceptable level of service, it is necessary that service volume is less than capacity. Service volume is thus the maximum volume that can be carried at any selected level of service.

The speed volume and volume density relationships are not same at various volume levels of traffic flow. These may be classified into the following levels of service:

(i) Free flow: Upto a volume level of 150 VPH, the operating speed is very near the free speed, density is low , and only less than 5 percent of vehicles are delayed. Thus upto this volume level , there is little or no restriction to the manoeuvrability due to other vehicles in the stream.

(ii) Stable flow: This zone lies between 150 to 650 VPH. Here manoeuvrability begins to be restricted by other vehicles due to interaction and the operating speed decreases linearly with volume. Yet steady flow is achieved and maintained.

(iii) Unstable flow: This zone lies between volume levels of above 650 to less than 700. There is a significant change in operating speeds and about 50 percent of the

vehicles are delayed. Volume speed relationship is nonlinear and this indicates that flow is approaching capacity.

(iv) At volume level of around 700 or more, the outgoing volume (or service volume) is less than incoming volume and this continues to decrease with time thereby increasing density and reducing operating speed. Speed and service volume may both drop to zero under jamming conditions.

The capacity of the 2 lane 2 way highway can thus be said to be between 650 to 700 VPH. However at a volume level of 700 , the outgoing volume rate reduces to 650 only after half an hour. Considering this to be an acceptable level of service, the capacity can be considered to be 700 vehicles per hour in either direction or a total of 1400. This may be compared with the capacity of 2000 specified by HRB (1965). The difference may be attributed to lower free speed of passenger cars in India and perhaps a differing stream logic. Various levels of service and their characteristics are presented in Fig. 6.4 and Table 6.2.

6.3.2 Homogeneous Truck Traffic

The general characteristics of homogeneous truck traffic are similar to those of homogeneous car traffic. Speed volume relationship for homogeneous truck obtained

TABLE 6.2 CHARACTERISTICS OF HOMOGENEOUS CAR TRAFFIC AT VARIOUS LEVELS OF SERVICE

Level of Service	Arriving Volume	Maximum Service volume (MSV)	$\frac{\text{MSV}}{\text{Capacity}}$	Operating Speed kmph
Free Flow	0-150	150	≤ 0.214	$50-0.0014\ V-0.000019\ V^2$
Stable Flow	150-650	650	≤ 0.857	$50.875-0.0101\ V$
Unstable Flow	650-700	≤ 700	≤ 1.00	Variable
Forced Flow	> 700	Widely Variable; Capacity Approaches Zero	$\ll 1.0$	May drop to nearly zero

from simulation is shown in Fig. 6.6. When traffic volume is low, i.e., upto 100 VPH , the available headways are large, there is hardly any constraint to manoeuvrability of vehicles and the operating speed of truck is very near the free speed. The speed decreases nonlinearly with volume and can be represented by;

$$OS_{Trucks} = 44.3 - 0.0009\ V - 0.000043\ V^2 \quad \text{for } V \leqslant 100 \qquad (6.9)$$

Between volume levels of 100 to 450 , the operating speed decreases due to smaller spacings and more interaction at higher volume levels. The operating speed of trucks in the zone of stable flow is given by;

$$OS_{Trucks} = 43.78 - 0.0143\ V \qquad (6.10)$$
$$\text{for } 100 \leqslant V \leqslant 450$$

The flow characteristics are unstable at volume levels between 450 to 500 VPH. At volume level of around 500 and beyond, the operating speed continues to decrease with time alongwith service volume. They may both drop to zero when traffic gets completely jammed. The capacity of the 2 lane 2 way highway may be considered to be around 475 trucks per hour for either direction or a total of 950. The various levels of service and their characteristics are summarised in Table 6.3

TABLE 6.3 CHARACTERISTICS OF HOMOGENEOUS TRUCK TRAFFIC AT VARIOUS LEVELS OF SERVICE

Level of Service	Arriving Volume	Maximum Service Volume (MSV)	MSV / Capacity	Operating Speed kmph
Free Flow	0 - 100	100	$\leq$ 0.211	$44.3-0.0009V-0.000043\ V^2$
Stable Flow	100 - 450	450	$\leq$ 0.947	$43.78-0.0143\ V$
Unstable Flow	450 -(< 500)	475	$\leq$ 1.0	Variable
Forced Flow	> 500	Widely Variable Capacity Approaches Zero	$\ll$ 1.0	May drop to nearly zero

6.4 Interaction Between Two Categories of Vehicles

6.4.1 Introduction

When traffic is composed of two categories of vehicles, i.e., passenger cars and those of another category, the flow characteristics like operating speed, density, delay etc. are affected by the following interactions:

(i) between cars themselves;

(ii) between cars and vehicles of the other category; and

(iii) between vehicles of the other category.

The manoeuvrability of cars is restrained by vehicles that move slower than cars. As the speed difference increases, the number of desired passing manoeuvres increase and they are further restrained by vehicles of the opposite stream. If a slow moving vehicle comes in the zone of conflict of an overtaking vehicle, it takes more time to cross the zone thereby restraining the overtaking vehicle and causing more delay. The interaction between the two categories of vehicles is likely to be affected by the composition of these vehicles in the mix. The interaction between slow moving vehicles is small as there is less of overtaking among them. The interaction between cars themselves was discussed in Subsec. 6.3.1. The maximum delay is likely

to be for cars due to their high speed. The operating speeds of cars at varying volume levels and proportions of the two vehicles in the mix were estimated from simulation.

6.4.2 Speed Volume Relationships for Different Combinations

Cars and Trucks: The free speed distributions of cars and trucks overlap over a sufficiently large extent as the difference between their mean free speeds is only 5.7 kmph. A number of trucks may thus have more speed than some of the cars. Trucks being longer need a larger minimum spacing. So car and truck traffic cannot be categorised as homogeneous.

Fig. 6.7 shows the operating speed of cars at various volume levels and for different car truck compositions. The operating speed decreases nonlinearly with volume and may be represented as follows:

$$OS_{MT2} = 57.7\ V^{-0.038} \quad \text{for} \quad 50 \leq V \leq 300 \qquad (6.11)$$

for car - truck ratio of 70/30

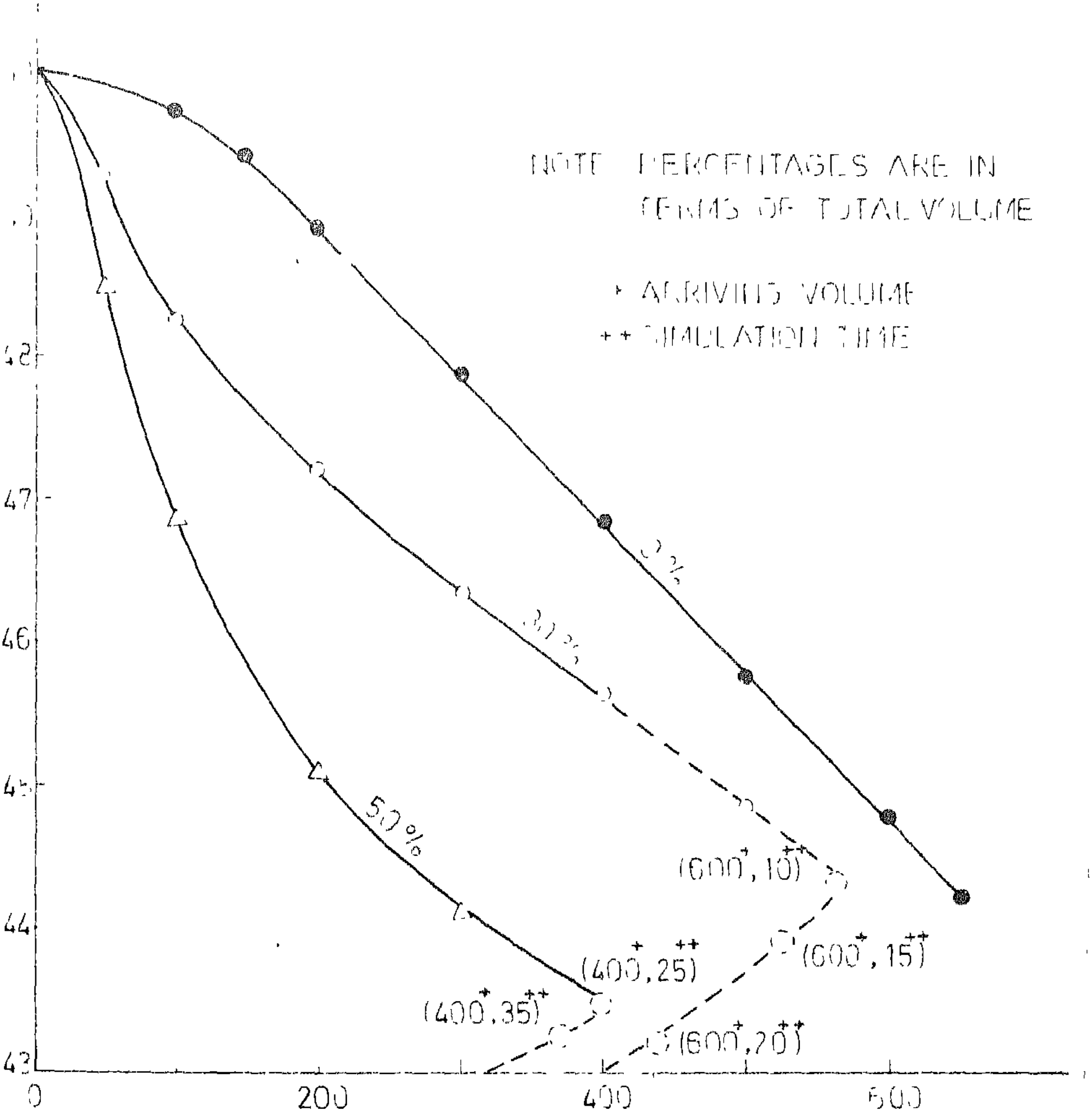
and $$OS_{MT2} = 59.8\ V^{-0.054} \quad \text{for} \quad 50 \leq V \leq 300 \qquad (6.12)$$

for car - truck ratio of 50/50

where OS_{MT2} = operating speed of cars in the mixed traffic

and V = total traffic volume .

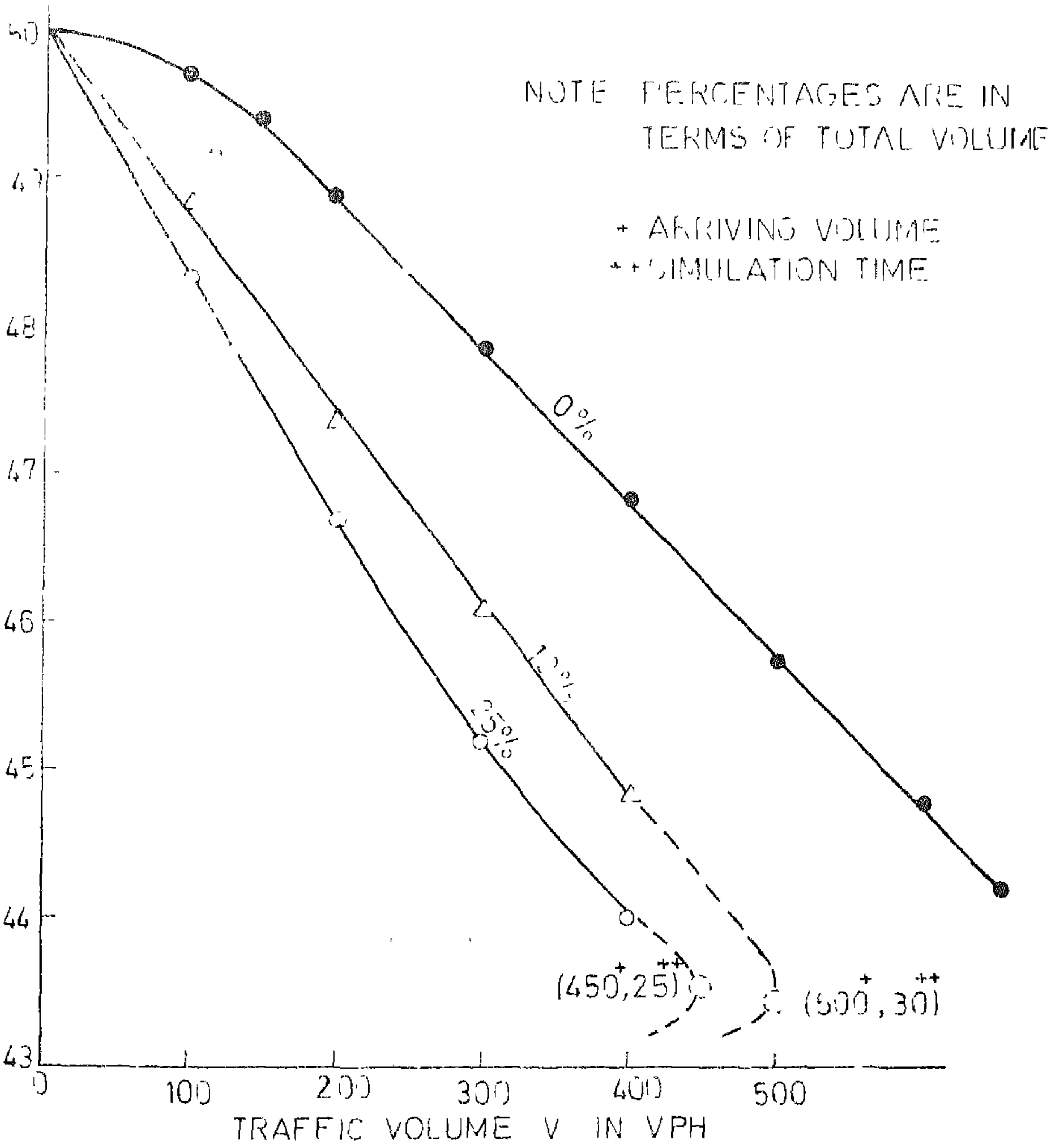
There is a considerable reduction in operating speeds even at low volume levels as interaction builds up quickly. Furthermore, for the some volume of total traffic,



.67 SPEED VOLUME RELATIONSHIP FOR CAR TRUCK COMBINATIONS

operating speed decreases with increased proportion of trucks in the mix as cars are restrained by a larger number of trucks in the stream. The operating speed of cars is thus a function of volume level and traffic composition. When there are 30 percent of trucks in the mix, the operating speed continues to decrease with time at a volume level of 600. Even after 10 minutes of simulation, service volume falls to 560 and decreases further alongwith operating speed. So for this composition, the road can carry per lane a maximum traffic volume of 500. With 50 percent of trucks, jamming starts occuring at a volume level of 400 after 25 minutes. Thus capacity is only 400 VPH for this composition.

Cars and Tongas: There is a wide difference in the free speed of cars and tongas and there is very little possibility of any delay to tongas. The main interactions are between cars themselves and between cars and tongas. Fig. 6.8 shows that for 10 percent of tongas, there is a linear relationship between the operating speed of cars and total traffic volume (V) upto 400 VPH. Beyond volume level of 400 , operating speed decreases nonlinearly and jamming starts occuring at a volume level of 500 after 30 minutes. When there are 25 percent tongas in the mix, the operating



G.6.8 SPEED VOLUME RELATIONSHIP FOR CAR TONGA COMBINATIONS

speed decreases nonlinearly with volume and jamming starts occuring at a volume of 450 VPH. The road can thus carry 500 and 450 VPH respectively with tonga proportions of 10 and 25 percent.

The speed volume relationships for the stable zone can be represented as:

$$OS_{MT2} = 50 - 0.0129\ V \quad \text{for} \quad V \leqslant 400 \qquad (6.13)$$

for 10 percent tongas

$$OS_{MT2} = 67.1\ V^{-0.069} \quad \text{for} \quad V \leqslant 400 \qquad (6.14)$$

for 25 percent tongas

Cars and Bullock Carts: Fig. 6.9 shows the speed volume relationships for varying compositions of bullock carts in the mix. Bullock cart, being the slowest vehicle causes maximum delay to the fast vehicles like cars. In case a car is unable to overtake, it has to reduce its speed to that of bullock cart, thereby considerably reducing its operating speed. The operating speed decreases exponentially with traffic volume. Even with only 10 percent bullock carts in the mix, the roadway can carry only 300 VPH and at a volume level of 350 , jamming builds up immediately. As the proportion of bullock carts increases, the operating speed and capacity are considerably reduced.

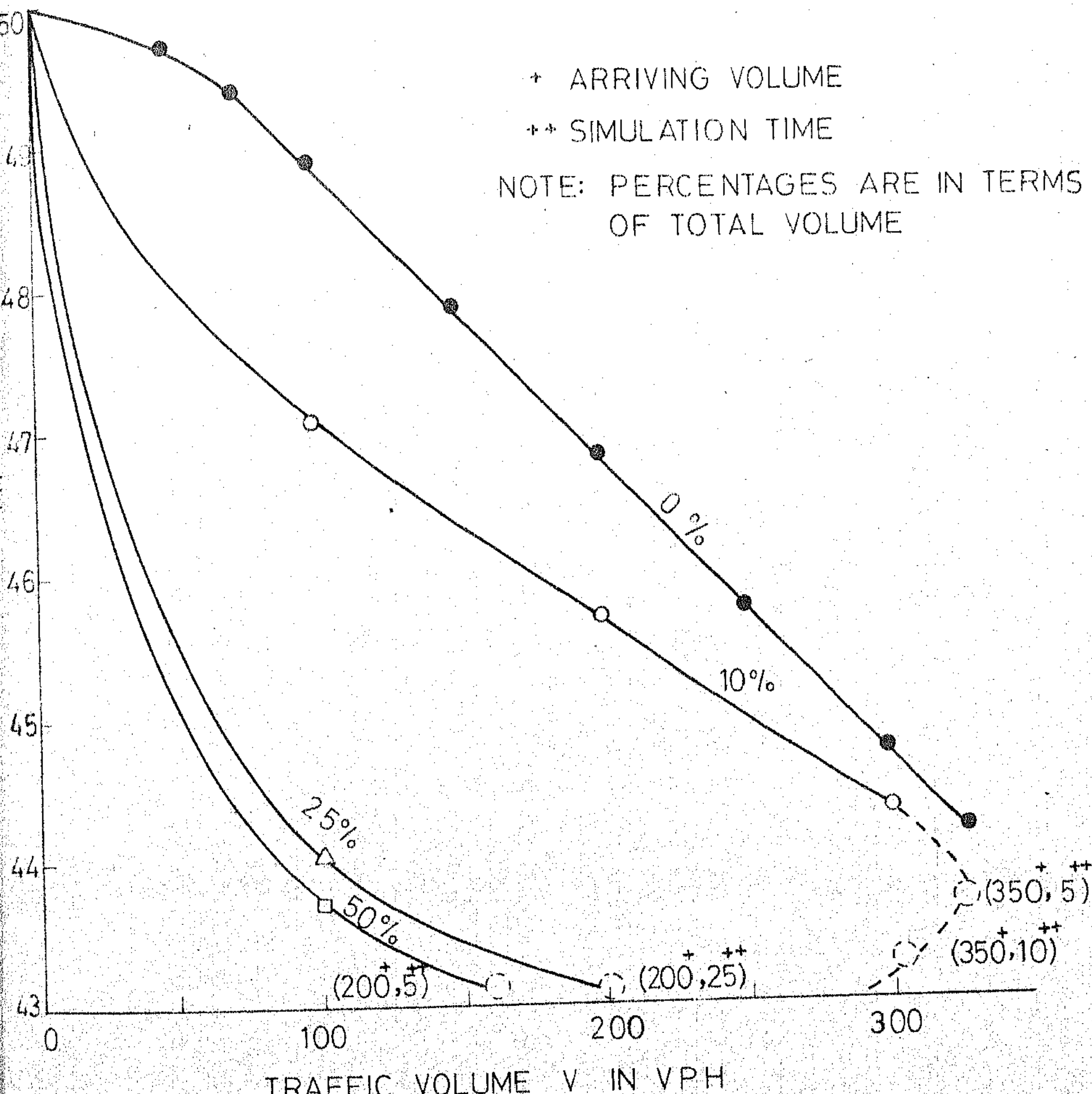


FIG 6·9 SPEED VOLUME RELATIONSHIP FOR CAR BULLOCK-CART COMBINATIONS

Cars and Scooters: The mean free speed of scooters and motorcycles is less than that of cars by 10.3 kmph and there is a very small overlap of their free speed distributions. Thus very few scooters may overtake cars. Scooter, is a two wheeler ,it occupies very little pavement width and its simulation logic (Sec. 5.3) is different from that of four wheelers. A car may overtake a scooter without moving into the wrong lane. However, if a scooter overtakes a car, it travels in the wrong lane during the overtaking operation. Fig. 6.10 shows that with 25 percent scooters in the mix, the operating speed is very close to the free speed upto a volume of 200 VPH. The operating speed of cars is given by;

$$OS_{MT2} = 50 - 0.0008V - 0.000012\ V^2 \qquad (\ 6.15\ )$$

$$\text{for} \quad V \leqslant 200$$

Beyond a volume level of 200 , the operating speed reduces linearly upto a level of 800, at which jamming starts building up after more than half an hour. The relationship is given by;

$$OS_{MT2} = 51.14 - 0.009\ V \qquad (\ 6.16\ )$$

$$\text{for } 200 \leqslant V \leqslant 800$$

When there are 50 percent scooters in the mix, free flow occurs upto a volume level of 250 and operating speed changes linearly from 250 to 900. Jamming occurs

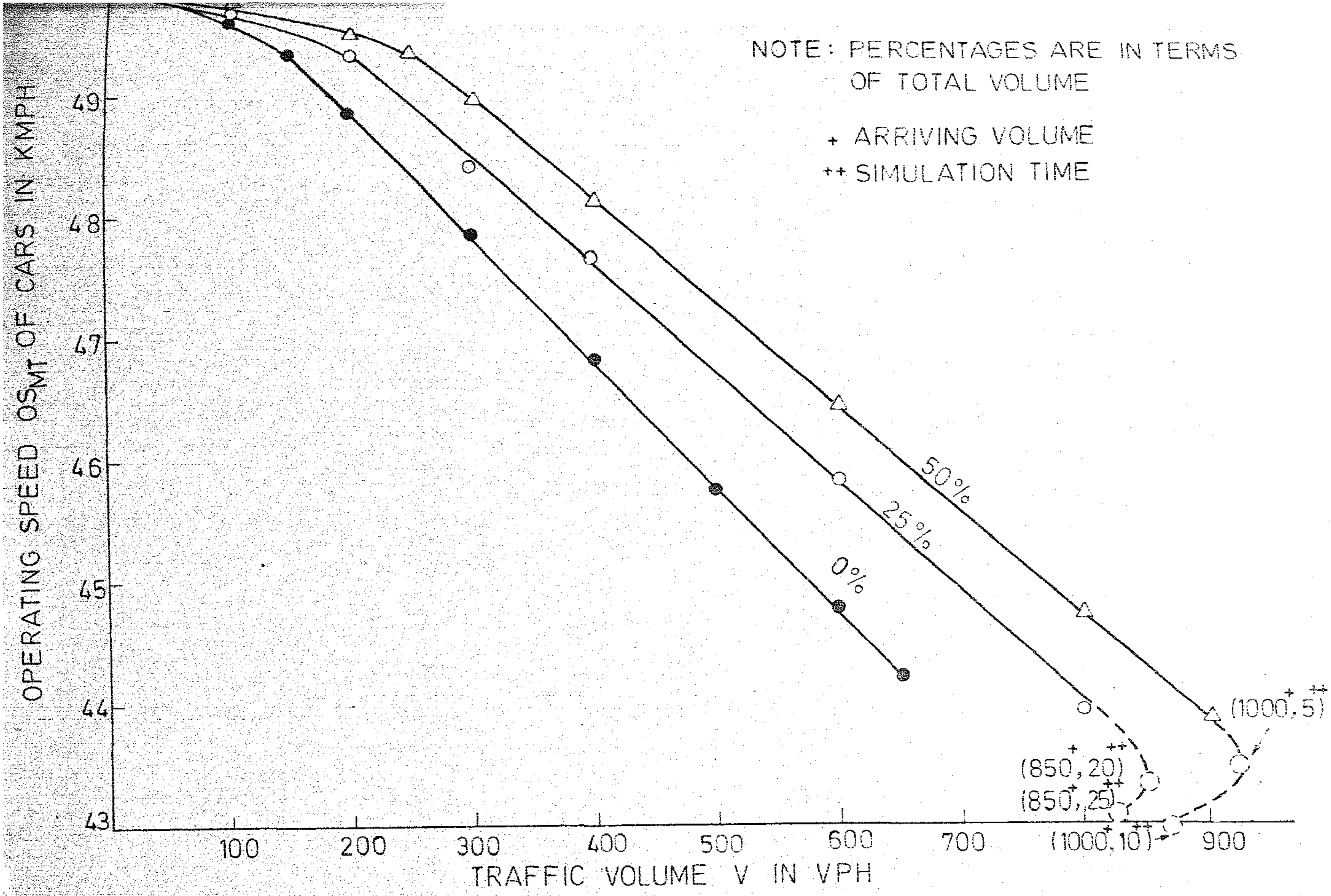


FIG.6.10 SPEED VOLUME RELATIONSHIP FOR CAR SCOOTER COMBINATIONS

immediately at volume level of 1000. The operating speed of cars is given by

$$OS_{MT2} = 50 - 0.0008 \ V - 0.000007 \, V^2 \qquad (6.17)$$

$$\text{for } V \leqslant 250$$

and

$$OS_{MT2} = 51.50 - 0.00856 \ V \qquad (6.18)$$

$$\text{for } 250 \leqslant V \leqslant 900$$

The capacity is 850 and 900 VPH respectively with 25 and 50 percent of scooters in the mix.

Fig 6.10 shows that operating speed of cars at a particular volume level is more with higher percentage of scooters in the mix. This is in contrast to that for the combination of cars with trucks, tongas or bullock carts (Fig. 6.7 to 6.9), in which operating speed of cars decreases with increased proportion of the second vehicle. With increased proportion of scooters in the total volume, there are a smaller number of cars and so there is less interaction between cars and scooters and between cars themselves. In a car truck combination, a higher proportion of trucks may reduce the interaction amongst cars; but there is considerable increase in the car - truck interaction and between trucks themselves. Consequently the operating speed of cars is considerably reduced.

Car and Bicycles: The bicycles are two wheelers and have simulation logic similar to scooters. The mean free speed of bicycles is much less than that of cars and there is no overlap of their free speed distributions. The main interaction is thus between the cars themselves. Fig. 6.11 shows that when there are 25 percent bicycles in the mix, the cars move under free flow conditions upto a volume of 200 VPH. The free flow is upto 250 and 300 VPH respectively with 50 and 75 percent bicycles in the mixed flow. The operating speeds of cars in the free flow are given by the following relationships:

$$OS_{MT2} = 50 - 0.0016\ V - 0.000009\ V^2 \qquad (6.19)$$

for $V \leq 200$

for 25 percent bicycles

$$OS_{MT2} = 50 - 0.0013\ V - 0.000006\ V^2 \qquad (6.20)$$

for $V \leq 250$

for 50 percent bicycles; and

$$OS_{MT2} = 50 - 0.0010\ V - 0.000005\ V^2 \qquad (6.21)$$

for $V \leq 300$

for 75 percent bicycles.

At higher volume levels the flow is stable and there is a linear reduction in operating speed with the volume level upto 800 VPH. The operating speed of cars in the stable zone is given by

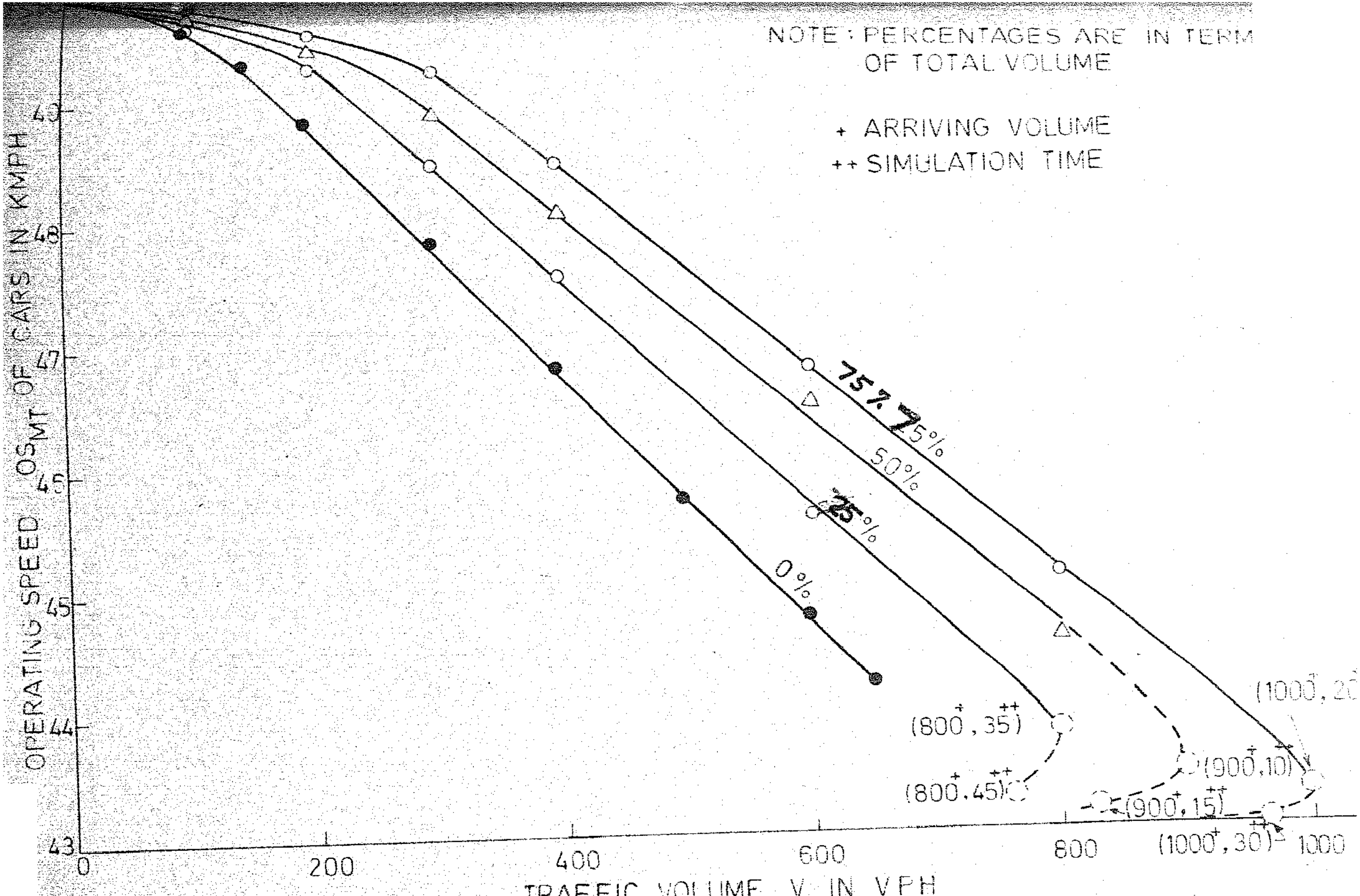


FIG-6.11 SPEED VOLUME RELATIONSHIP FOR CAR BICYCLE COMBINATIONS

$$OS_{MT2} = 51.13 - 0.00905\ V \quad \text{for} \quad 200 \leq V \leq 800 \qquad (6.22)$$

for 25 percent bicycles

$$OS_{MT2} = 51.39 - 0.00836\ V \quad \text{for} \quad 250 \leq V \leq 850 \qquad (6.23)$$

for 50 percent bicycles ; and

$$OS_{MT2} = 51.71 - 0.0082\ V \quad \text{for} \quad 300 \leq V \leq 900 \qquad (6.24)$$

for 75 percent bicycles.

Fig. 6.11 shows that operating speed of cars at a particular volume level is more with higher proportion of bicycles in the mix. Maximum service volumes that approach capacity are 800 , 850 and 1000 VPH respectively at 25 , 50 and 75 percent of bicycles in the mix.

The above results demonstrate clearly that interaction is a function of volume level and traffic composition . Mathematical equations derived may be valid only for specific conditions and compositions. Interaction between cars themselves have been estimated in Subsec. 6.3.1. When there are two vehicles, viz., cars and vehicles of another category, there is a need to estimate the interaction between:

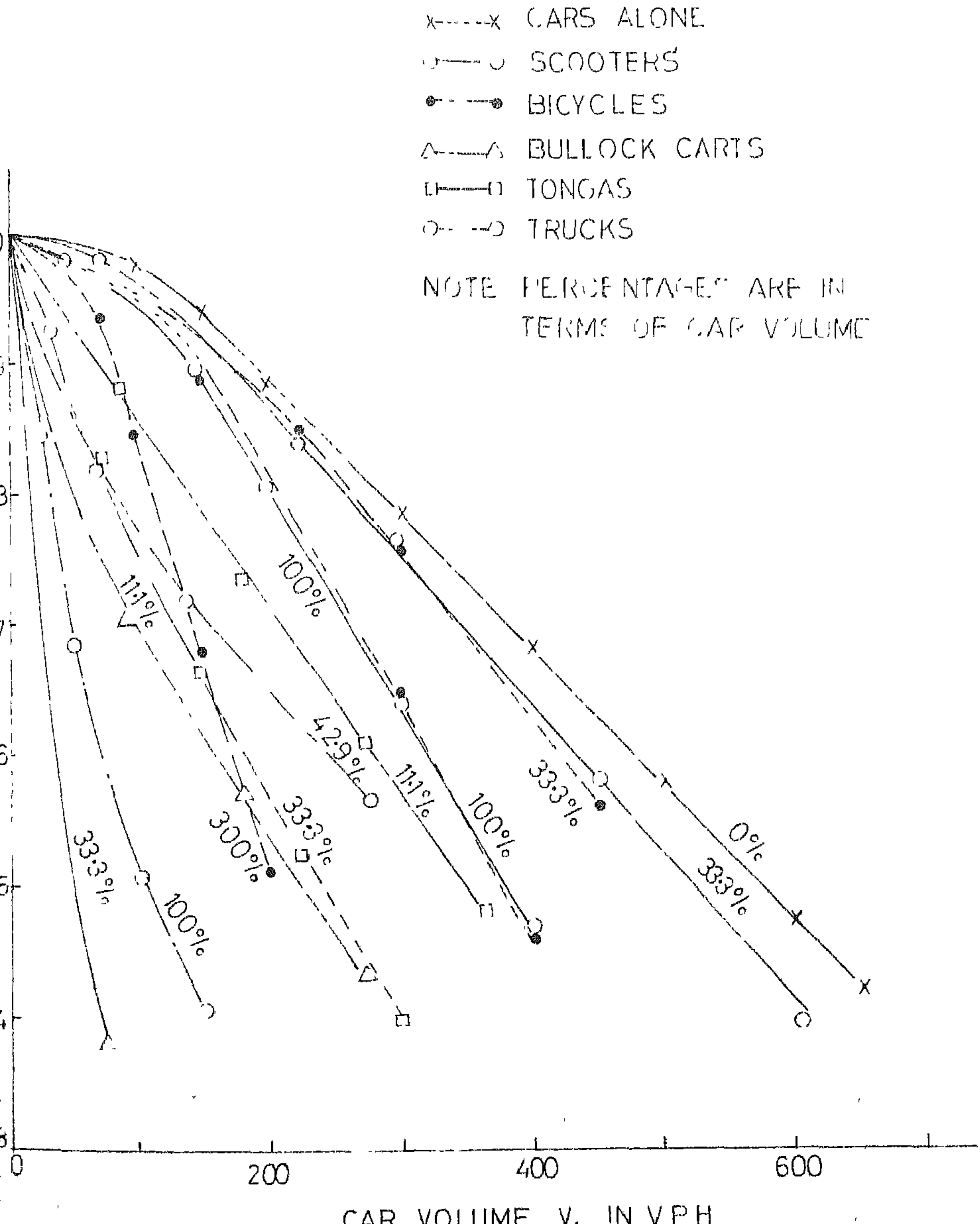
(i) cars and vehicles of the other category like trucks, tongas, bullock carts, scooters and bicycles; and

(ii) among the second category of vehicles themselves.

These interactions reduce the operating speed from that for homogeneous car traffic. The change in operating speed is a function of the volume of cars and the proportion of the second category expressed as percentage of car volume . Hence Fig. 6.12 shows the operating speed volume relationship for cars for different compositions of the second category of vehicles. These may be compared with Figs. 6.7 to 6.11 where the relationships are in terms of total volume of traffic. The effect of interactions is more clearly evident in Fig. 6.12. Equations were fitted to define the speed volume relationships of cars for different compositions of two vehicle combinations and are given in Table 6.4. It may be noted that in Fig. 6.12, the plotted points are derived from simulation and plotted curves correspond to fitted relationships (Table 6.4).

6.4.3 Maximum Service Volumes for Different Combinations

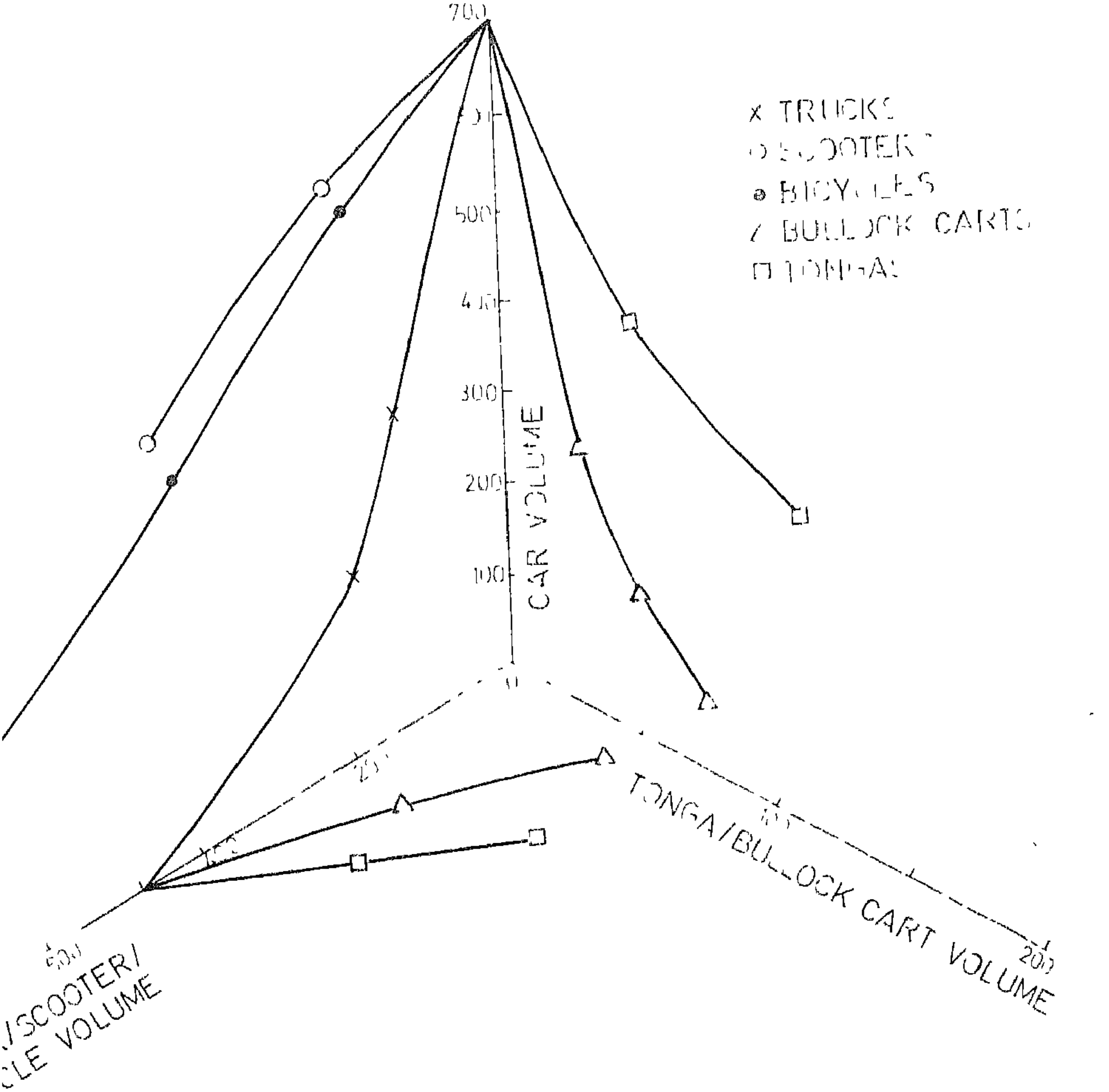
For a given composition of two vehicle combination, there is a maximum possible service volume which defines the roadway capacity. Five curves each for a two vehicle combination are shown in Fig. 6.13 and these define the capacity of a two lane two way highway. Car scooter and car bicycle curves are very close to each other at the various compositions. Maximum service volume is a little



…6·12 SPEED VOLUME RELATIONSHIP OF CARS FOR DIFFERENT TWO VEHICLE COMBINATIONS

TABLE 6.4 SPEED VOLUME RELATIONSHIPS OF CARS FOR TWO VEHICLE COMBINATIONS

Combination	Proportion of Second Vehicle as Percentage of Car Volume	Operating Speed of Cars (OS_{MT2}) kmph	
Cars Alone	–	$50-0.0014\ V_1-0.000019\ V_1^2$	for $V_1 \leq 150$
		$50.875-0.0101\ V_1$	for $150 \leq V_1 \leq 650$
		V_1 = Car Volume	
Cars and Trucks	42.9	$57.6\ V_1^{-0.0405}$	for $35 \leq V_1 \leq 280$
	100.0	$57.6\ V_1^{-0.0525}$	for $25 \leq V_1 \leq 150$
Cars and Tongas	11.1	$50-0.01433\ V_1$	for $V_1 \leq 360$
	33.3	$56.7\ V_1^{-0.0427}$	for $V_1 \leq 300$
Cars and Bullock Carts	11.1	$58.2\ V_1^{-0.047}$	for $V_1 \leq 270$
	33.3	$58.2\ V_1^{-0.065}$	for $V_1 \leq 75$
Cars and Scooters	33.3	$51.14-0.012\ V_1$	for $150 \leq V_1 \leq 600$
	100.0	$51.50-0.0171\ V_1$	for $125 \leq V_1 \leq 400$
Cars and Bicycles	33.3	$51.13-0.01207\ V_1$	for $150 \leq V_1 \leq 600$
	100.0	$51.39-0.0167\ V_1$	for $125 \leq V_1 \leq 400$
	300.0	$51.71-0.0328\ V_1$	for $75 \leq V_1 \leq 225$



G·6·13 MAXIMUM SERVICE VOLUME FOR DIFFERENT COMPOSITIONS OF TWO VEHICLE COMBINATIONS

more for car scooter combination than car bicycle combination at the same composition. This indicates a larger interaction between cars and bicycles than between cars and scooters. For vehicles of wide base and for the same proportion, maximum service volume increases as the relative speed between the car and the second vehicle decreases, viz., from car bullock carts combination through car - tonga combination to car truck combination.

6.4.4 A Multiplicative Model for Speed Volume Relationship of Mixed Traffic Flow

The results of simulation analyses indicate that for a given volume of cars, the operating speed of cars decreases with increased proportion of the second category of vehicles. The interaction factor, IF , represents the ratio of operating speed of cars in mixed traffic (OS_{MT2}) to the operating speed of cars in homogeneous traffic (OS_{HT}), i.e.,

$$IF = \frac{OS_{MT2}}{OS_{HT}} \qquad (6.25)$$

The operating speed of cars in mixed flow can be represented as:

where V_1 = volume of cars in the mixed flow; P_2 = proportion of the second vehicle expressed as a percentage of V_1 ; and CAT = the second category of vehicles in mixed flow.

The interaction factor can also be represented by

$$IF = f_2 (V_1 , P_2 , CAT) \quad (6.27)$$

From Eq. 6.25 and 6.27

$$OS_{MT2} = OS_{HT} (V_1) . f_2 (V_1, P_2 , CAT) \quad (6.28)$$

Hence the operating speed of cars in mixed traffic is a product of the operating speed of cars alone and the interaction factor, IF , which depends upon the volume of cars (V_1), and the type (CAT) and proportion (P_2) of vehicles of second category. This is referred to as a multiplicative model for speed volume relationship in the mixed traffic flow. It may be noted that the multiplicative model is designed to agree with relationships for homogeneous and mixed traffic flows.

From the results of simulation analysis given in Table 6.4 and Fig. 6.12, the following relationships were developed for different combinations of vehicles.

Let $P_.$ represent the percentage of vehicle of category (.) in the two vehicle combination. Let subscript T , KK , BC , SC and BY represent respectively trucks, tongas, bullock carts, scooters and bicycles.

Then for;

(i) Car - truck combination:

$$IF = 1.0 - 0.0012\,P_T + (0.0023 + 0.00112\,P_T)(10)^{-\frac{V_1}{(103 + 0.4666P_T)}} \qquad (6.29)$$

(ii) Car - tonga combination:

$$IF = 1.0 - (0.020 + 0.036\,P_{KK})[\,1 + 10^{-\frac{V_1}{(293 + 13.86\,P_{KK})}}\,] \qquad (6.30)$$

(iii) Car - bullock cart combination:

$$IF = 1.0 - 0.009\,P_{BC} + 0.0062\,P_{BC}[(10)^{-\frac{V_1}{(230 + 26.1\,P_{BC})}}\,] \qquad (6.31)$$

(iv) Car - scooter combination:

$$IF = 1.007 - (0.0007 + 0.000043\,P_{SC})[\,(10)^{\frac{V_1}{(700 - 2.8\,P_{SC})}}\,] \qquad (6.32)$$

(v) Car - bicycle combination:

$$IF = 1.0 - [\,-0.0015 + 0.000126\left(\frac{P_{BY}}{1 + 0.01\,P_{BY}}\right)(10)^{\frac{V_1}{(486 - 0.975P_{BY})}}\,] \qquad (6.33)$$

Though simulation analysis performed in this study is fairly extensive it had to be limited to only a few proportions for each combination because of limitations of computer time. Yet it indicates clearly the nonlinearity

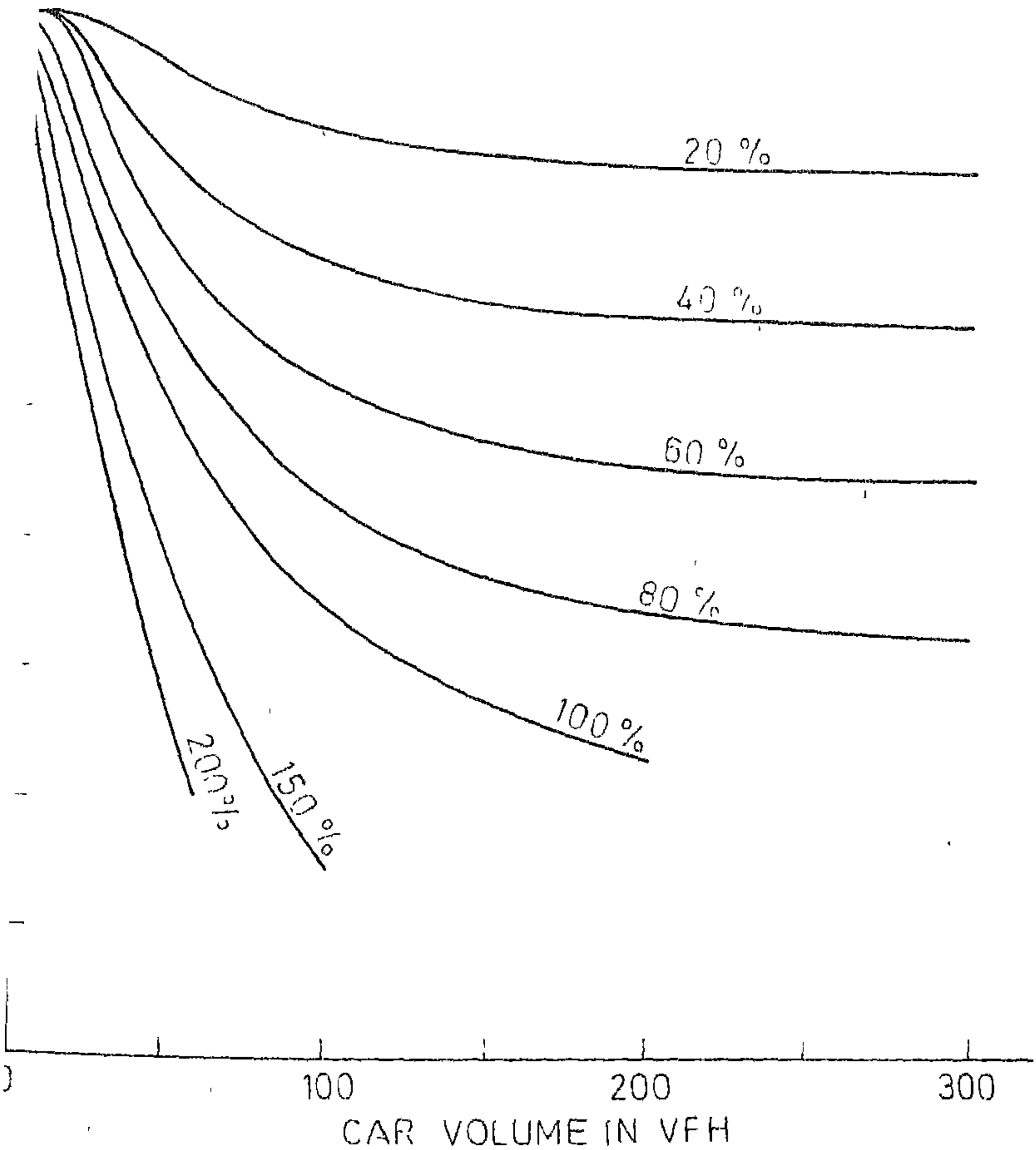
of the interactions and furthermore there is a general consistency in the results. Hence Eqs. 6.29 to 6.33 may be considered satisfactory even though further refinement of parameters and equations for IF is possible.

Using the Eqs. 6.29 to 6.33 , the interaction factor IF for different combinations and for various proportions were derived and they are shown in Figs. 6.14 to 6.18. They may be used as design charts for traffic having characteristics similar to those considered in this study. In the absence of additional information, it may be presumed that , IF is a function of V_1 , P_2 and CAT only, and is independent of the free speed distribution of vehicles. In such a case, the speed volume relationship of cars alone for any other free speed distribution of cars can be established and used with the design charts (Figs. 6.14 to 6.18) to determine the speed volume relationship for mixed traffic flow.

6.5 Interaction Between Three Categories of Vehicles

6.5.1 Introduction

Simulation was carried out for combinations of three categories of vehicles and was limited to car, truck and one of the remaining four categories, viz., tonga, bullock cart, scooter and bicycle. In such combinations, the flow



14 INTERACTION FACTOR FOR TRUCKS ON CARS

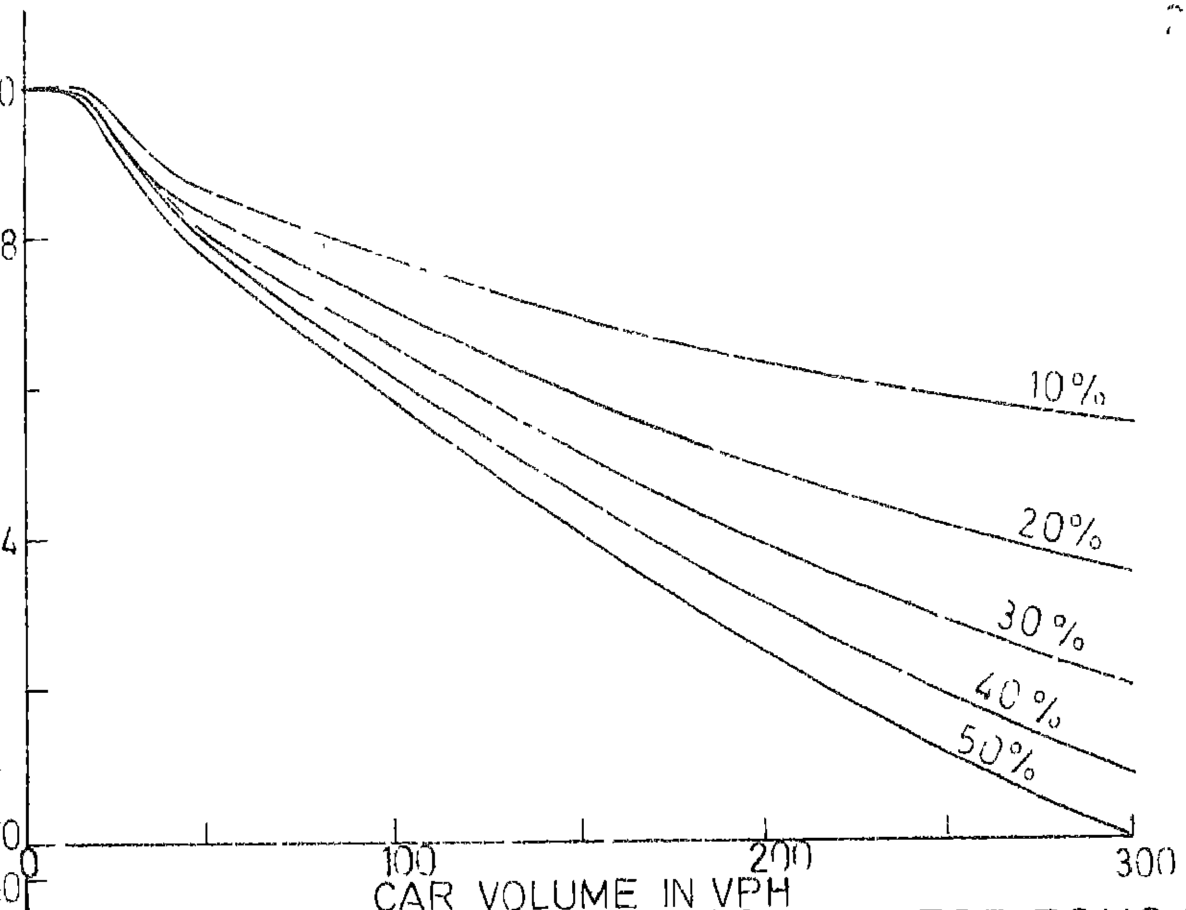
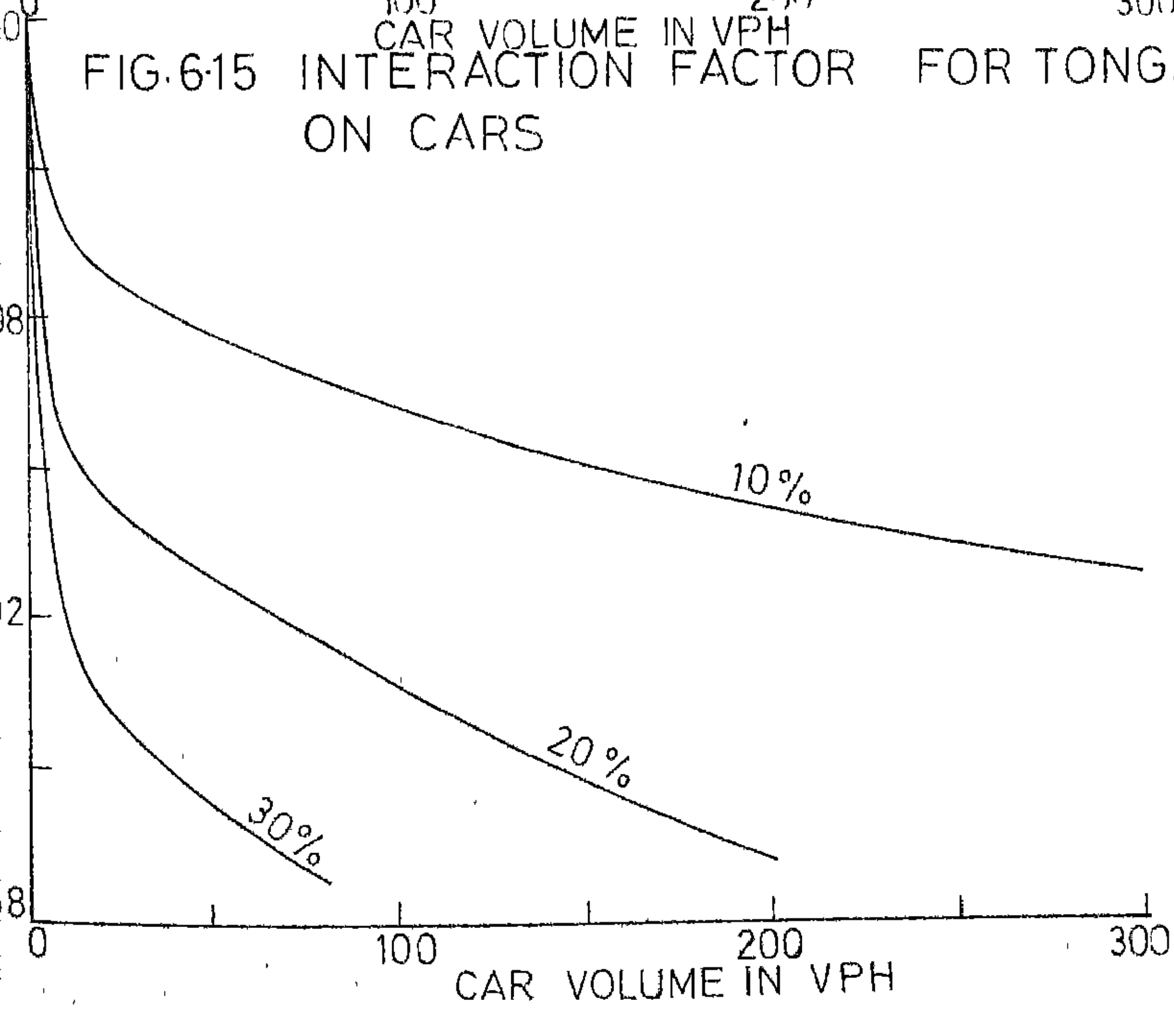
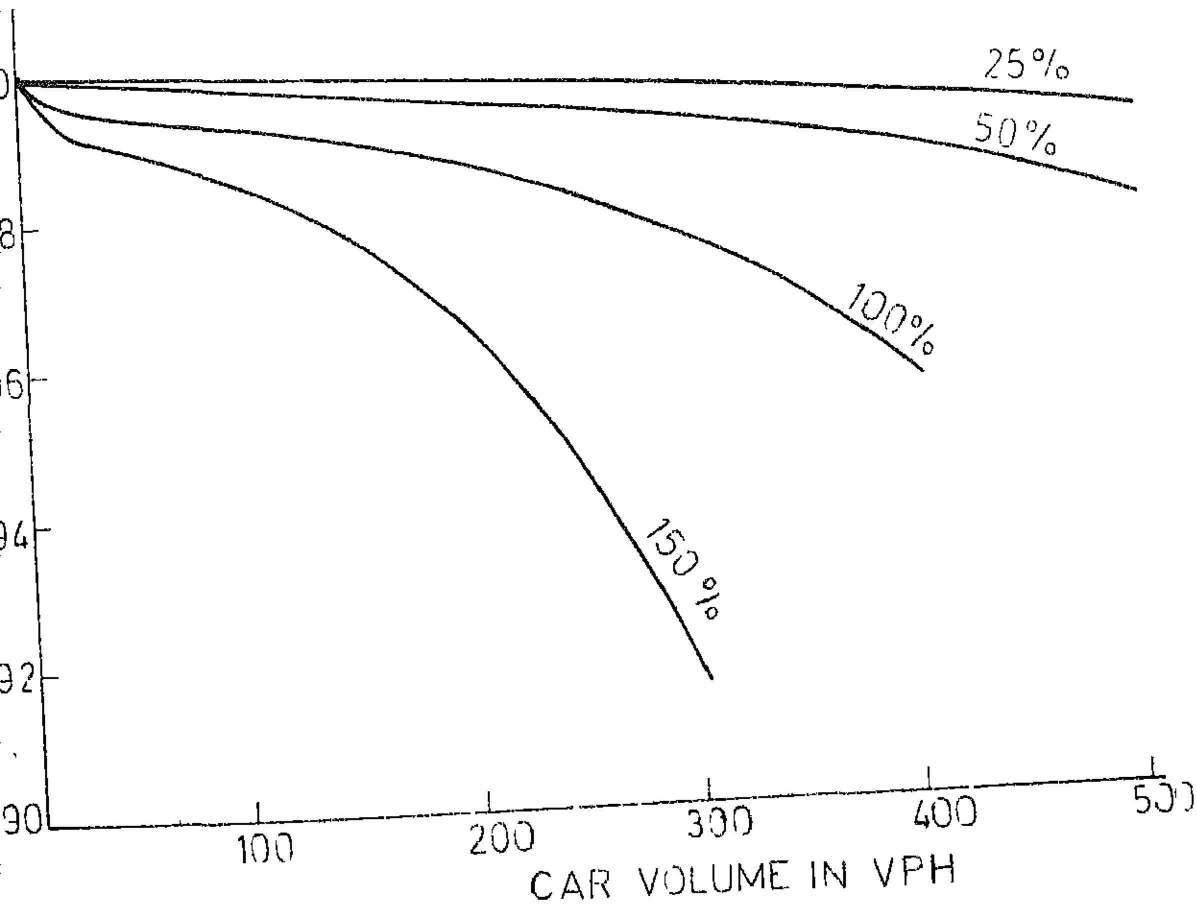


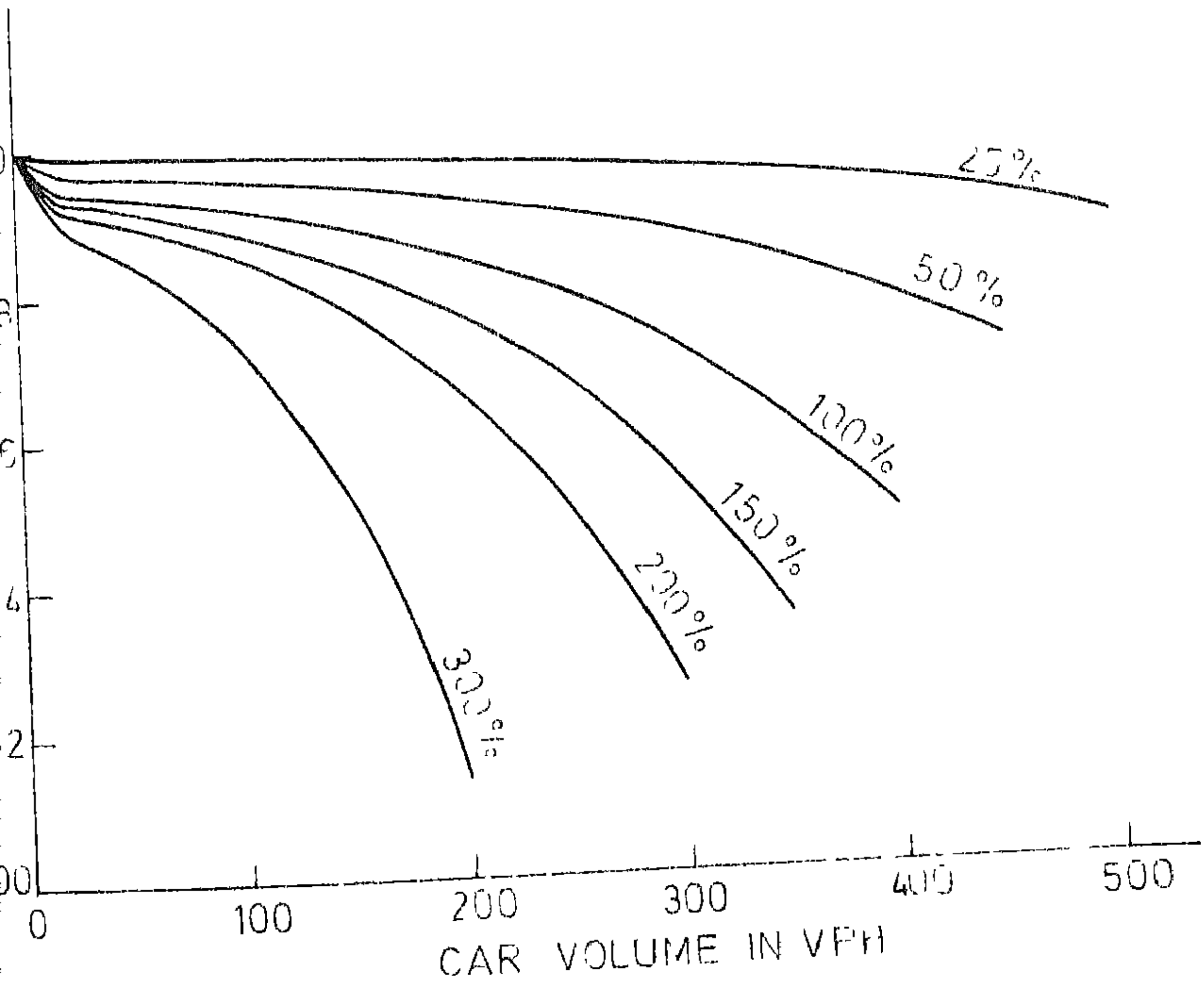
FIG. 6.15 INTERACTION FACTOR FOR TONGAS ON CARS



6.16 INTERACTION FACTOR FOR BULLOCK CARTS ON CARS



3·17 INTERACTION FACTOR FOR SCOOTERS ON CARS



6·18 INTERACTION FACTOR FOR BICYCLES ON CARS

characteristics are affected by the following:

(i) Interaction between cars themselves and trucks themselves;

(ii) Interaction between cars and trucks;

(iii) Interaction of vehicles of third category among themselves; and

(iv) Interaction between vehicles of third category with cars and trucks.

6.5.2 Speed Volume Relationships for Different Combinations

The flow characteristics were estimated by simulation for each combination at different compositions and volume levels (Table 6.1). Figs 6.19 to 6.22 show the operating speed of cars as a function of total traffic volume for different compositions of various three vehicle combinations. The results indicate that in case bullock carts or tongas are present alongwith cars and trucks , the operating speed of cars decreases with volume due to smaller headways and large number of constraints. Furthermore, for the same volume level the operating speed decreases with increased proportion of bullock carts or tongas, indicating a significant interaction between slow and fast moving vehicles. The operating speed also decreases with increased proportions

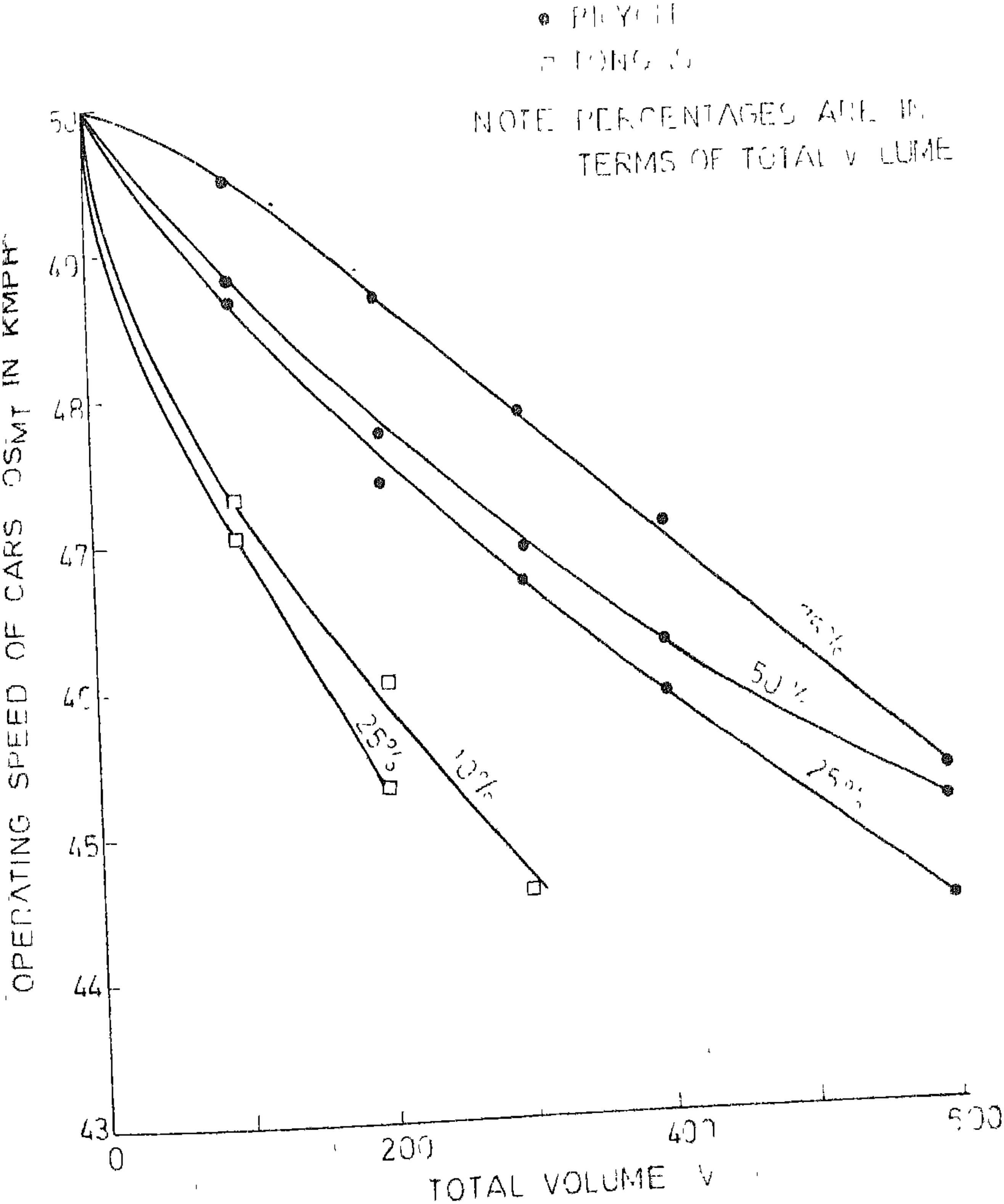


FIG. 6.19 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR TRUCK RATIO 70/30

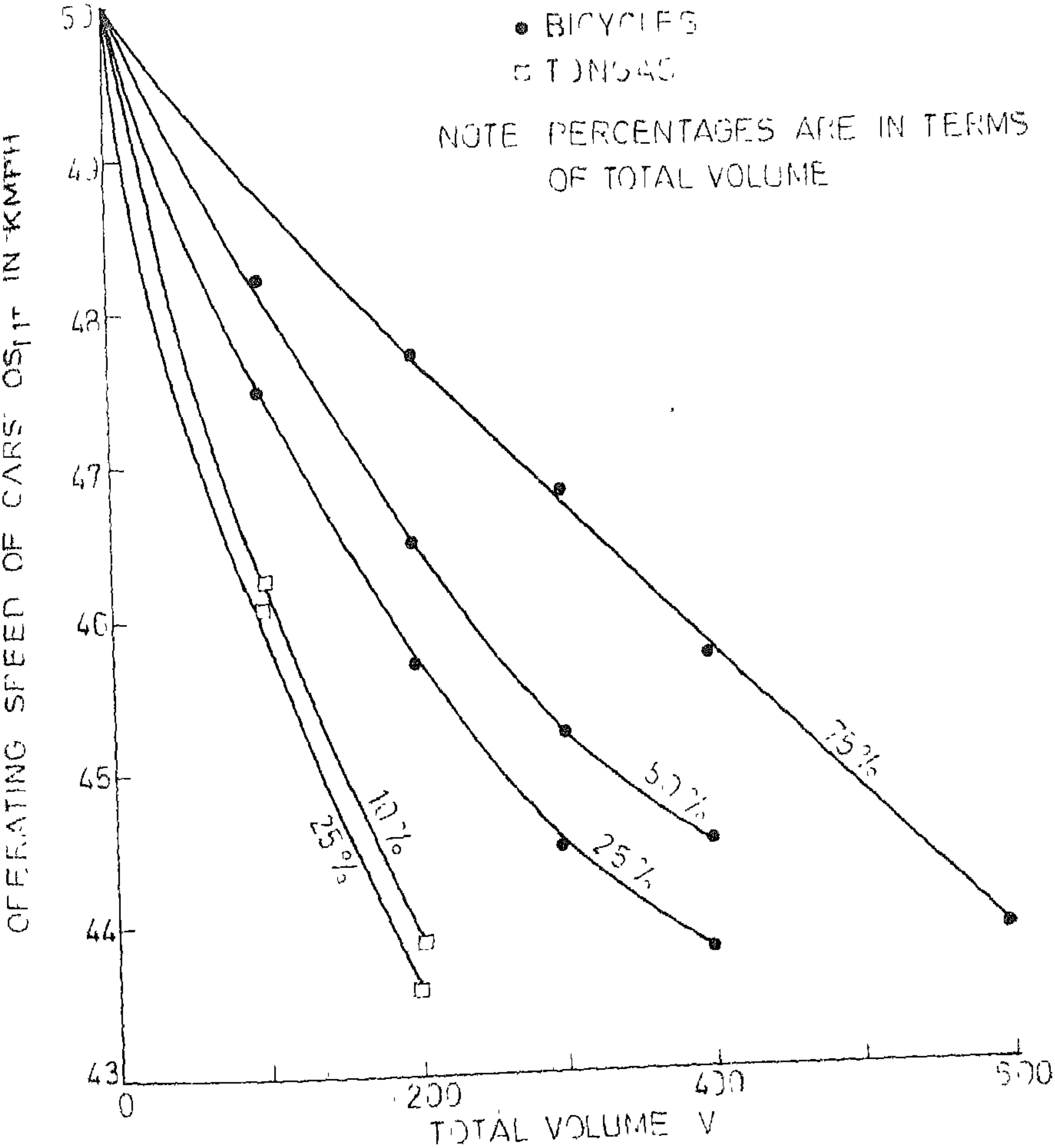


FIG.620 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR-TRUCK RATIO 50/50

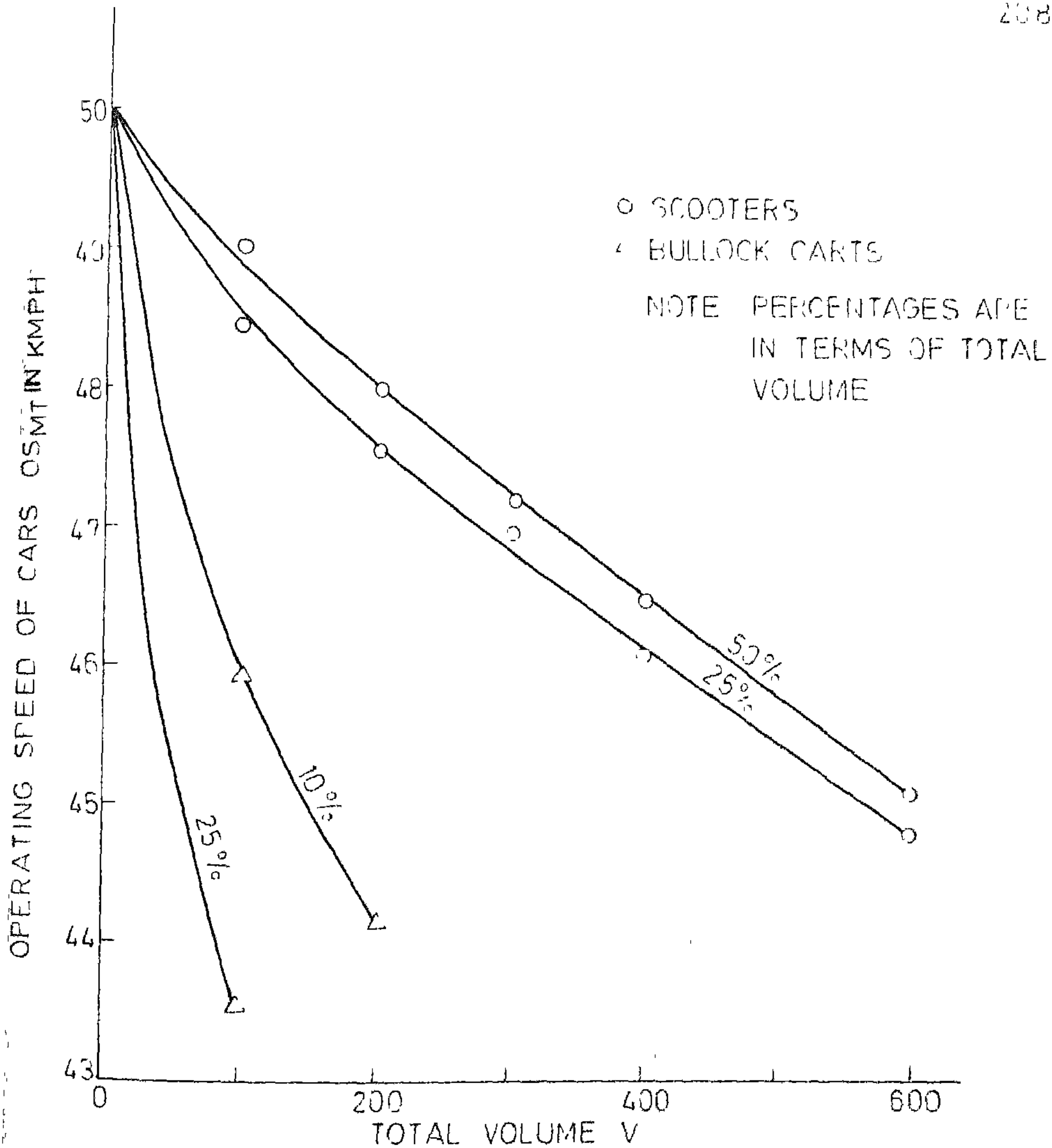


FIG.6.21 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR-TRUCK RATIO 70/30

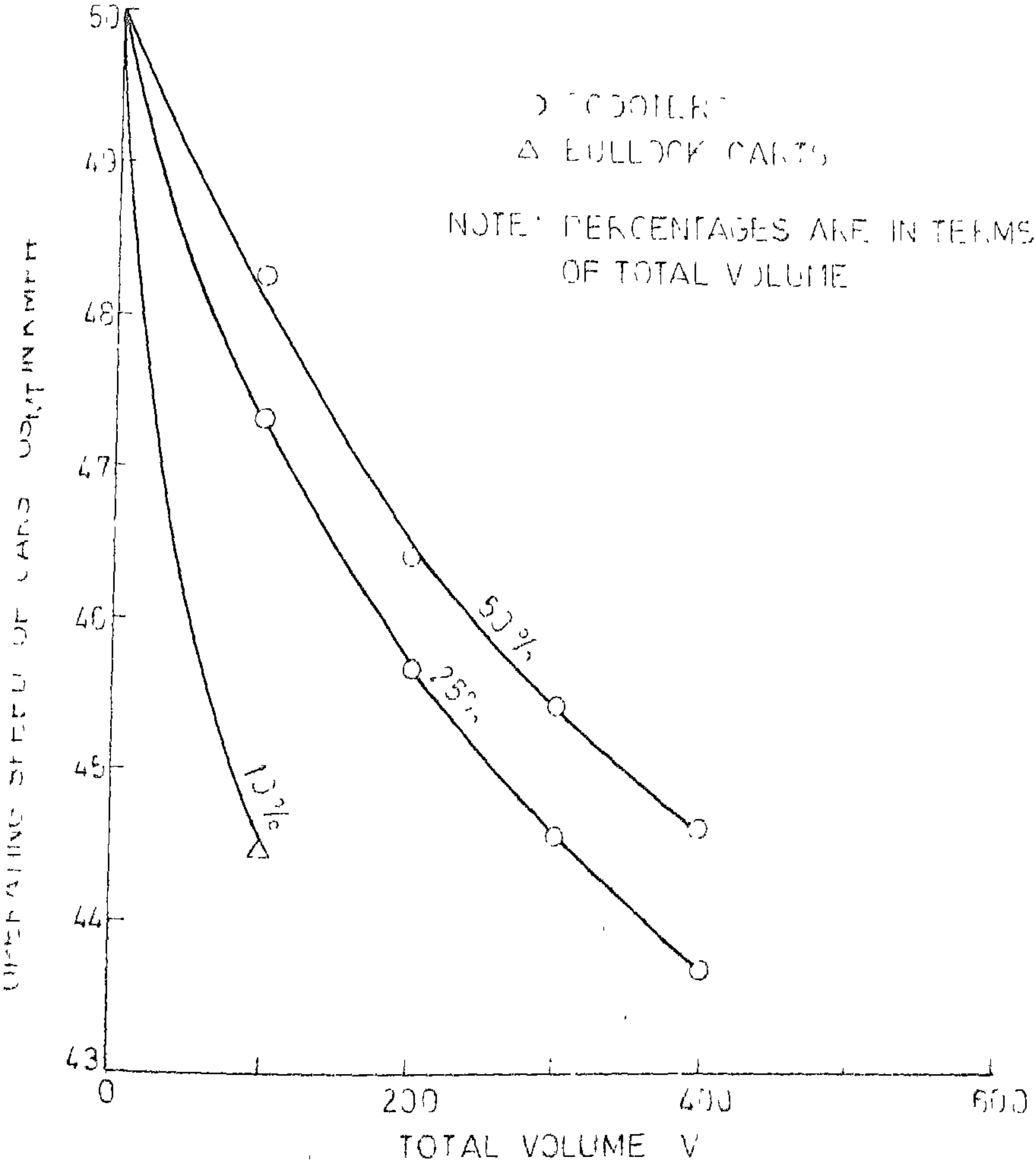


FIG. 6.22 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR-TRUCK RATIO 50/50

of trucks in the mix.

When the third category of vehicle is scooters or bicycles, the operating speed is less with a smaller proportion of scooters or bicycles. This is because two wheelers like scooters or bicycles occupy very less width and interaction with fast vehicles is not very significant. The results agree with those of two vehicle combinations. The speed volume relationships are generally exponential and they are not reported in this study.

6.5.3 Feasibility and Levels of Service for Three Vehicle Combinations

The operating speed of cars varies with volume of traffic, the combination of vehicles and their composition. In this study, it is proposed to define the level of service in terms of mean operating speed of cars. Hence for homogeneous traffic, the level of service worsens as the volume of cars increases, and in mixed traffic, for the same volume of cars, the level of service worsens with the proportion of other categories of vehicles. Different combinations and compositions of vehicles may represent the same level of service and so they may be connected by isolines or isosurfaces of same level of service.

The level of service may be considered as the response of the system to different combinations of vehicles and for three vehicle combination, it can be plotted in an isometric graph. The plots for three vehicle combinations with bicycles and bullock carts as the third vehicle , are shown respectively in Figs. 6.23 and 6.24. It may be noted that the feasible combinations of traffic are bounded by the three planes and an upper concave surface defining the capacity of a two lane two way highway. The surface defining capacities is bounded by its intersections with the bounding planes, which in turn define the capacity curves for two vehicle combinations.

The level surfaces for car truck bicycle combination for operating speeds of 48 , 46 and 44 kmph are shown in Fig. 6.23. It may be noted that level surfaces are also concave from above and may be bounded by the bounding planes or the capacity surfaces. For car truck bullock cart combination, only the level surface for 46 kmph and level curves of 48 and 44 kmph are shown in Fig. 6.24 to avoid confusion. They are similar to those of bicycles in Fig. 6.23 , but the level surfaces , curves and capacity surfaces for bullock carts are much steeper than in case of bicycles.

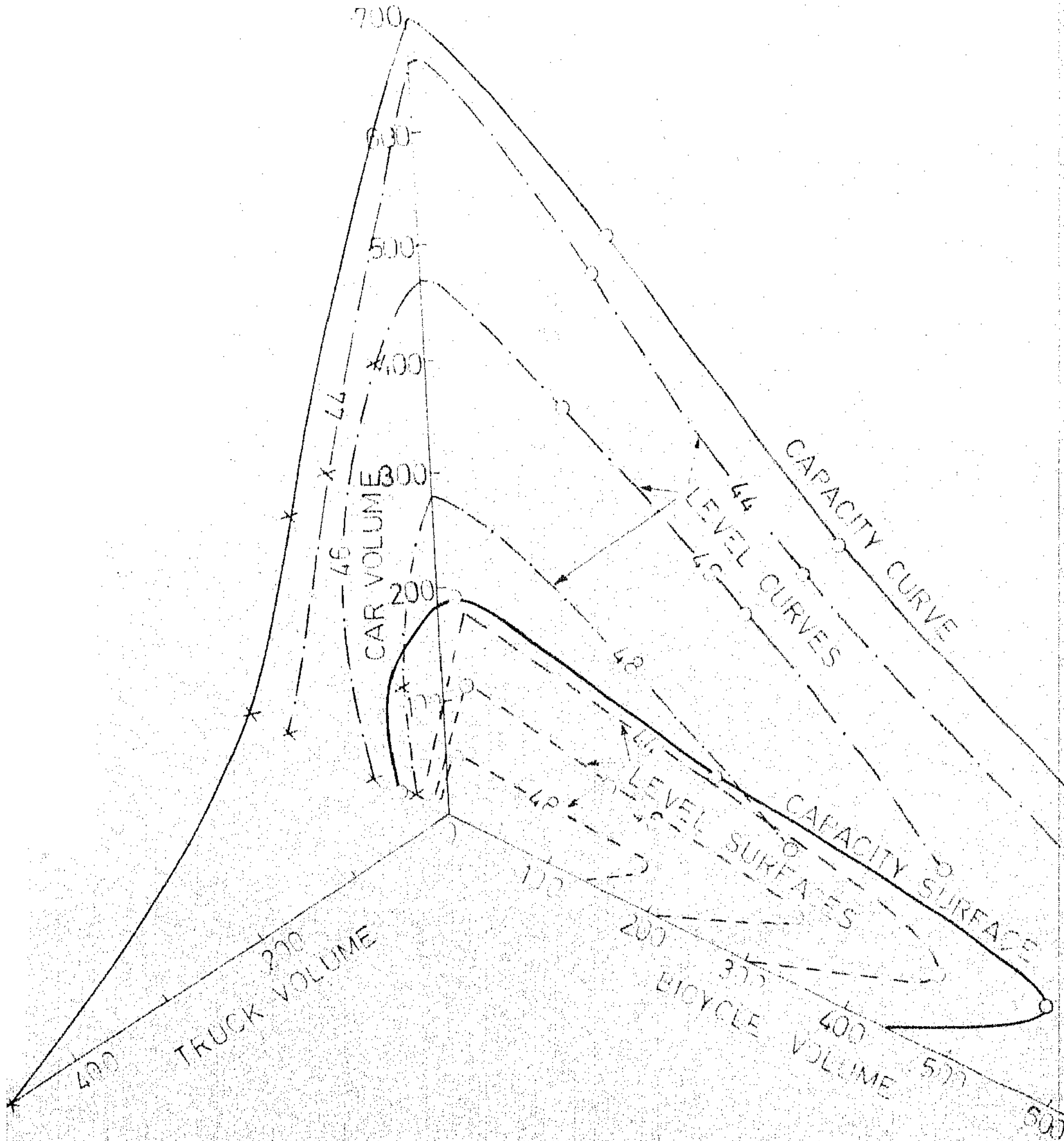
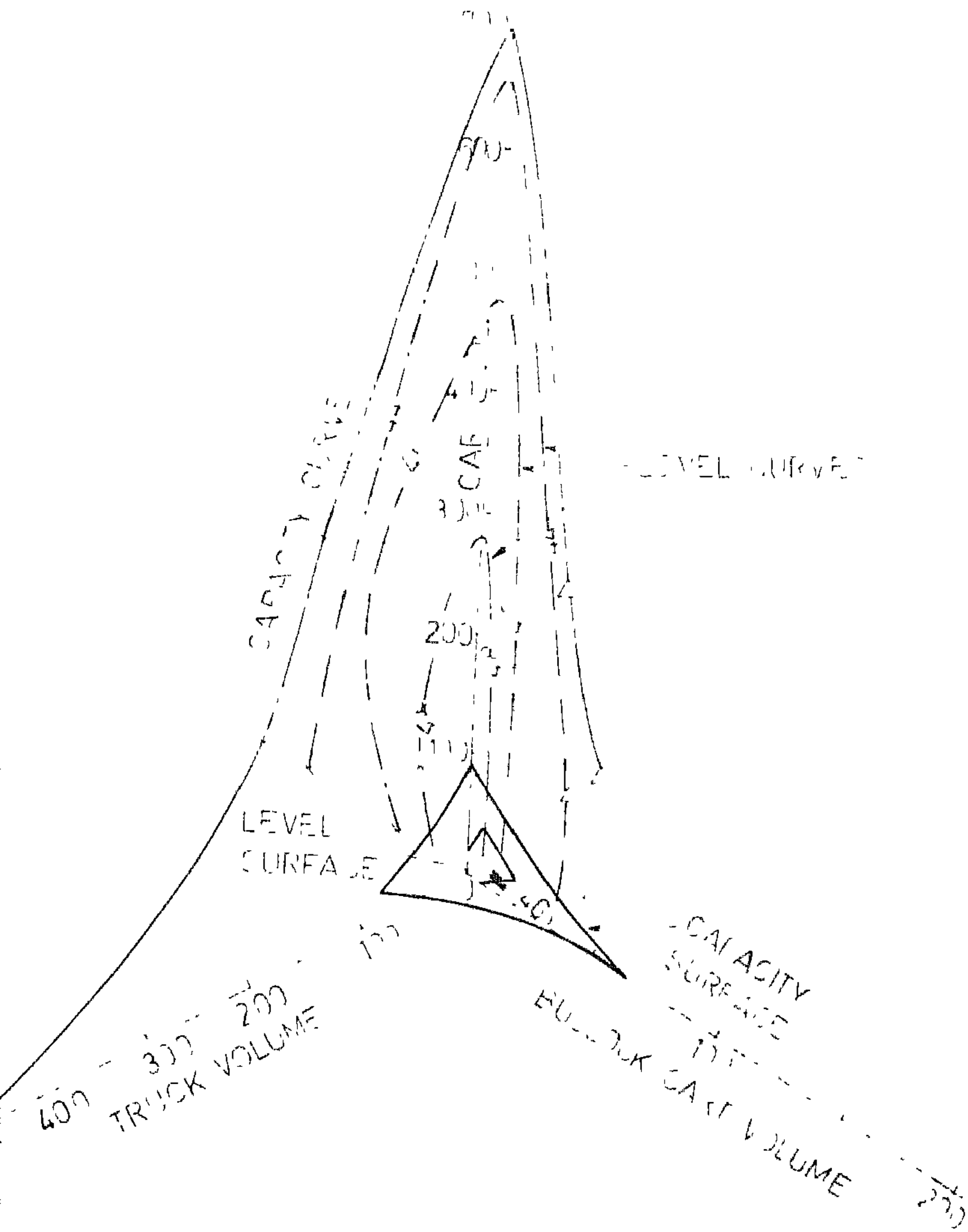


FIG. 6.23 LEVEL SURFACES FOR CAR TRUCK AND BICYCLE COMBINATIONS



6·24 LEVEL SURFACES FOR CAR TRUCK AND BULLOCK CART COMBINATIONS

6.5.4 Conditional Multiplicative Model

Speed volume relationships indicate that the general characteristics of three vehicle combinations are similar to those of two vehicle combinations, for which a multiplicative model has been developed. The operating speed of cars for different combinations depends upon the traffic composition and volume level of each category. Based on Eq. 6.26 , it can be expressed as ;

$$OS_{MT3} = f (V_1 , P_2 , P_3 , CAT) \qquad (6.34)$$

where V_1 = car volume in the total mix; and P_2 and P_3 are proportions (expressed as percentage of car volume) of trucks and vehicles of third category (CAT) respectively.

Assuming that the interactions of trucks and the third category of vehicles on cars are independent of each other, the multiplicative model was generalised as follows:

$$OS_{MT3} = OS_{HT} (V_1) . IF_2 (V_1 , P_2) . IF_3 (V_1, P_3, CAT) \qquad (6.35)$$

where IF_2 represents the interaction factor for trucks on cars and IF_3 , the interaction factor for third category of vehicles on cars. They are estimated from Eqs. 6.29 to 6.33.

The operating speed of cars for different three vehicle combinations and at different compositions and volume levels, were obtained from simulation and were compared

with the values calculated by Eq. 6.35. The results were not consistent indicating that the interaction between trucks and third category of vehicles is also significant. Hence the multiplicative model (Eq. 6.35) was modified as a conditional multiplicative model given by

$$OS_{MT3} = OS_{HT} (V_1) . IF_2 (V_1, P_2) . IF_3 [(V_1+V_2), P_3^{'}; CAT] \quad (6.36)$$

where $P_3^{'}$ is the proportion of the third category of vehicle in terms of car truck volume ($V_1 + V_2$) , and IF_3 is the conditional interaction factor for the third category of vehicles on the fast moving vehicles, cars and trucks.

Assuming that IF_3 has the same parameters and forms as in Eqs. 6.29 to 6.33 , but with V_1 replaced by car truck volume ($V_1 + V_2$) , the operating speed of cars in mixed traffic were calculated using Eq. 6.36 for different three vehicle combinations at different proportions and volumes. A sample calculation is shown in Table 6.5. They were compared with results from simulation analysis. (Table 6.6 and Figs. 6.25 to 6.28). The errors in the operating speed of cars has an average absolute value of around 0.2 percent with a maximum of around 0.5 percent. There is thus a very good agreement between simulation results and the fitted model confirming the validity of

TABLE 6.5 SAMPLE CALCULATIONS FOR OPERATING SPEED OF CARS IN MIXED VEHICULAR TRAFFIC

(i) Traffic composition : scooters = 25 % ;
Car truck ratio = 70/30;

Hence Cars = 52.5 % ; Trucks = 22.5 %
and Scooters = 25.0 %

(ii) Traffic Volume (V) = 200
Car Volume(V_1) = 105 ;
Truck Volume(V_2) = 45 ;
Scooter Volume(V_3) = 50

(iii) Operating speed of cars for homogeneous traffic (OS_{HT}) at volume V_1 of 105 VPH (Eq. 6.5) = 49.64 kmph

(iv) Percentage P_2 of trucks in terms of car volume
= 30/70 = 42.9 %

(v) Interaction factor, IF_2 for trucks on cars (Eq. 6.29)
= 0.9593

(vi) Percentage P_3' of scooters in terms of car truck volume
= 25/75 = 33.3 %

(vii) Interaction factor, IF_3 , for scooters on car truck combination (Eq. 6.32) = 0.9969

(viii) Operating speed of cars (OS_{MT3}) in mixed flow
$= OS_{HT} * IF_2 * IF_3$
= 47.43

(ix) From simulation results, OS_{MT3} = 47.52

(x) Error = 47.43 - 47.52 = -0.09
= 0.189 %

TABLE 6.6 COMPARISON OF SIMULATION RESULTS WITH THOSE COMPUTED FROM MULTIPLICATIVE MODEL FOR THREE VEHICLE COMBINATIONS (EQ. 6.36)

Combination	Composition in Percent	Volume	Operating Speed of Cars (OS_{1T_3}) kmph From Model	Operating Speed of Cars (OS_{1T_3}) kmph From Simulation	Error
Cars, Trucks and Tongas	Cars =63.0, Trucks =27.0, Tongas =10.0	100	47.27	47.33	-0.06
		200	45.85	46.01	-0.16
		300	44.45	44.56	-0.11
	Cars =52.5, Trucks =22.5, Tongas =25.0	100	47.23	47.05	0.18
		200	45.13	45.27	-0.14
	Cars =45.0, Trucks =45.0, Tongas =10.0	100	46.20	46.25	-0.05
	Cars =37.5, Trucks =37.5, Tongas =25.0	100	46.32	46.18	0.14
Cars, Trucks and Bullock Carts	Cars =63.0, Trucks =27.0, Bullock carts=10	100	45.75	45.94	-0.19
		200	44.04	44.15	-0.11
	Cars =45.0, Trucks =45.0, Bullock Carts =10.0	100	44.71	44.50	0.21
Cars, Trucks and Scooters	Cars =52.5, Trucks =22.5, Scooters=25.0	100	48.52	48.46	0.06
		200	47.43	47.52	-0.09
		300	46.72	46.98	-0.26
		400	46.03	46.10	-0.07
		600	44.75	44.80	-0.05
	Cars =35.0, Trucks =15.0, Scooters=50.0	100	48.90	49.06	-0.16
		200	47.81	48.02	-0.19
		300	47.07	47.21	-0.14
		400	46.46	46.51	-0.05
		600	45.14	45.09	-0.05

Contd...

TABLE 6.6 CONTD...

Combination	Composition in Percent	Volume	Operating Speed of Cars (OS_{MT_3}) kmph From Model	From Simulation	Error
Cars, Trucks and Scooters	Cars =37.5	100	47.58	47.33	0.25
	Trucks =37.5	200	45.76	45.69	0.07
	Scooters =25.0	300	44.61	44.57	0.04
		400	43.82	43.70	0.12
	Cars =25.0	100	48.24	48.25	-0.01
	Trucks =25.0	200	46.61	46.44	0.17
	Scooters =50.0	400	44.48	44.65	-0.17
Cars, Trucks and Bicycles	Cars =52.5	100	48.51	48.65	-0.14
	Trucks =22.5	200	47.41	47.40	0.01
	Bicycles =25.0	300	46.68	46.69	-0.01
		400	45.96	45.93	0.03
		600	44.51	44.41	0.10
	Cars =35.0	100	48.87	48.82	0.05
	Trucks =15.0	200	47.77	47.75	0.02
	Bicycles =50.0	300	47.02	46.96	0.06
		400	46.40	46.22	0.18
		600	44.99	45.11	-0.12
	Cars =17.5	100	49.41	49.48	-0.07
	Trucks = 7.5	200	48.48	48.66	-0.18
	Bicycles =75.0	300	47.68	47.85	-0.17
		400	46.93	47.08	-0.15
		600	45.32	45.28	0.04
	Cars =37.5	100	47.57	47.48	0.09
	Trucks =37.5	200	45.74	45.73	0.01
	Bicycles =25.0	300	44.58	44.49	0.09
		400	43.75	43.77	-0.02
	Cars =25.0	100	48.21	48.22	-0.01
	Trucks =25.0	200	46.58	46.49	0.09
	Bicycles =50.0	300	45.36	45.23	-0.13
		400	44.41	44.48	-0.07
	Cars =12.5	200	47.83	47.71	0.12
	Trucks =12.5	300	46.75	46.82	0.07
	Bicycles =75.0	400	45.76	45.73	0.03
		600	43.73	43.89	-0.16

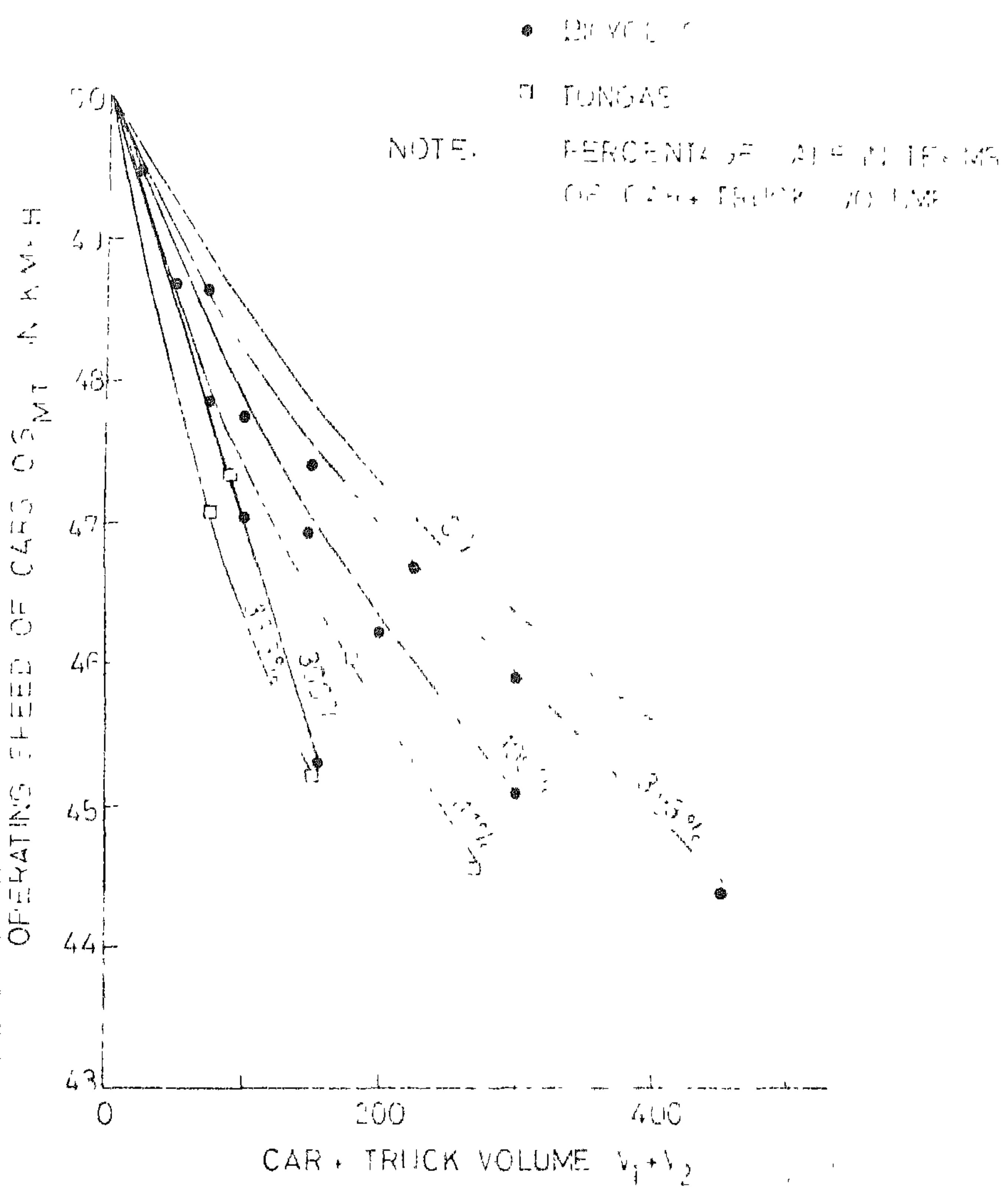


FIG 6·25 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR TRUCK RATIO 70/30

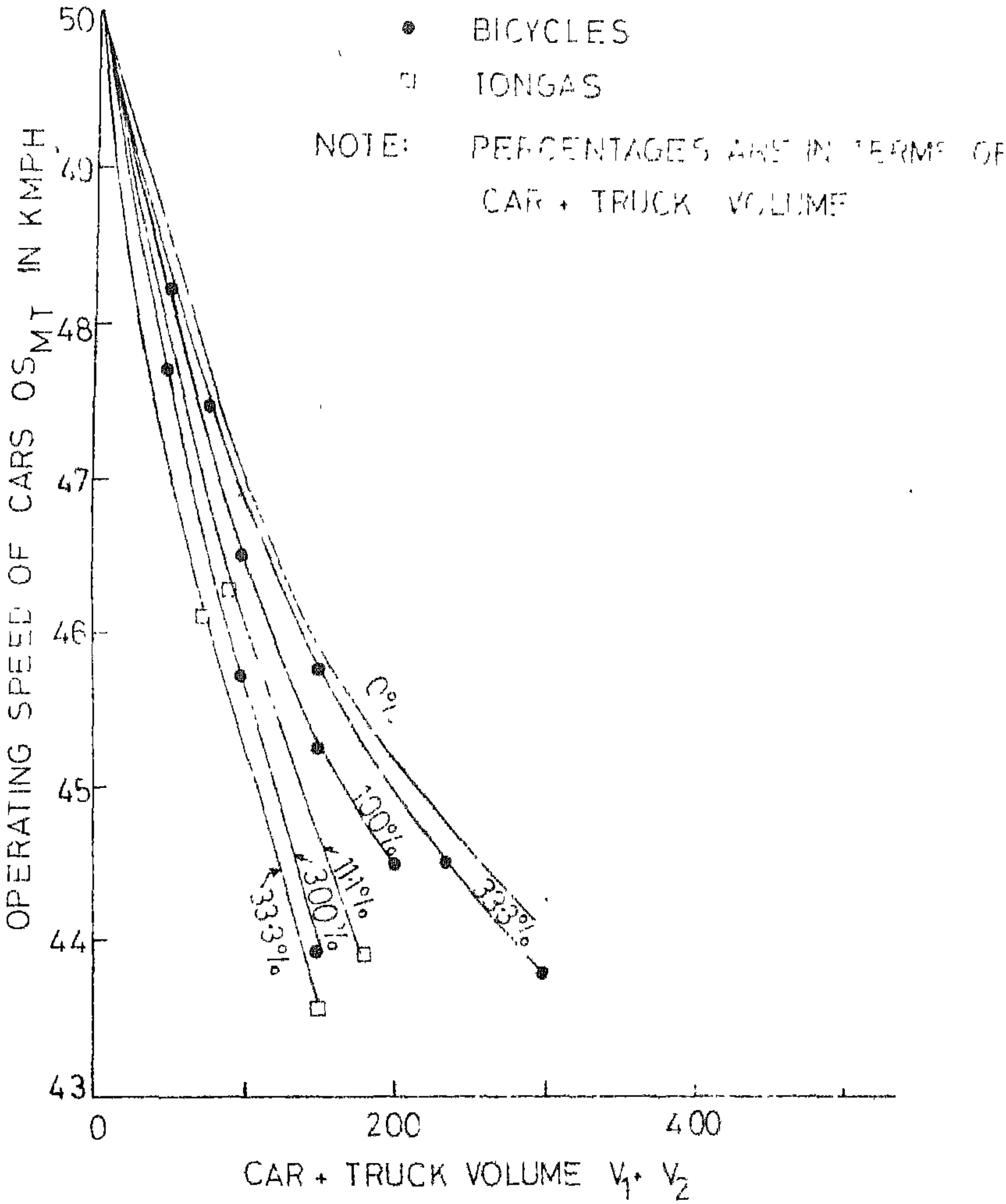
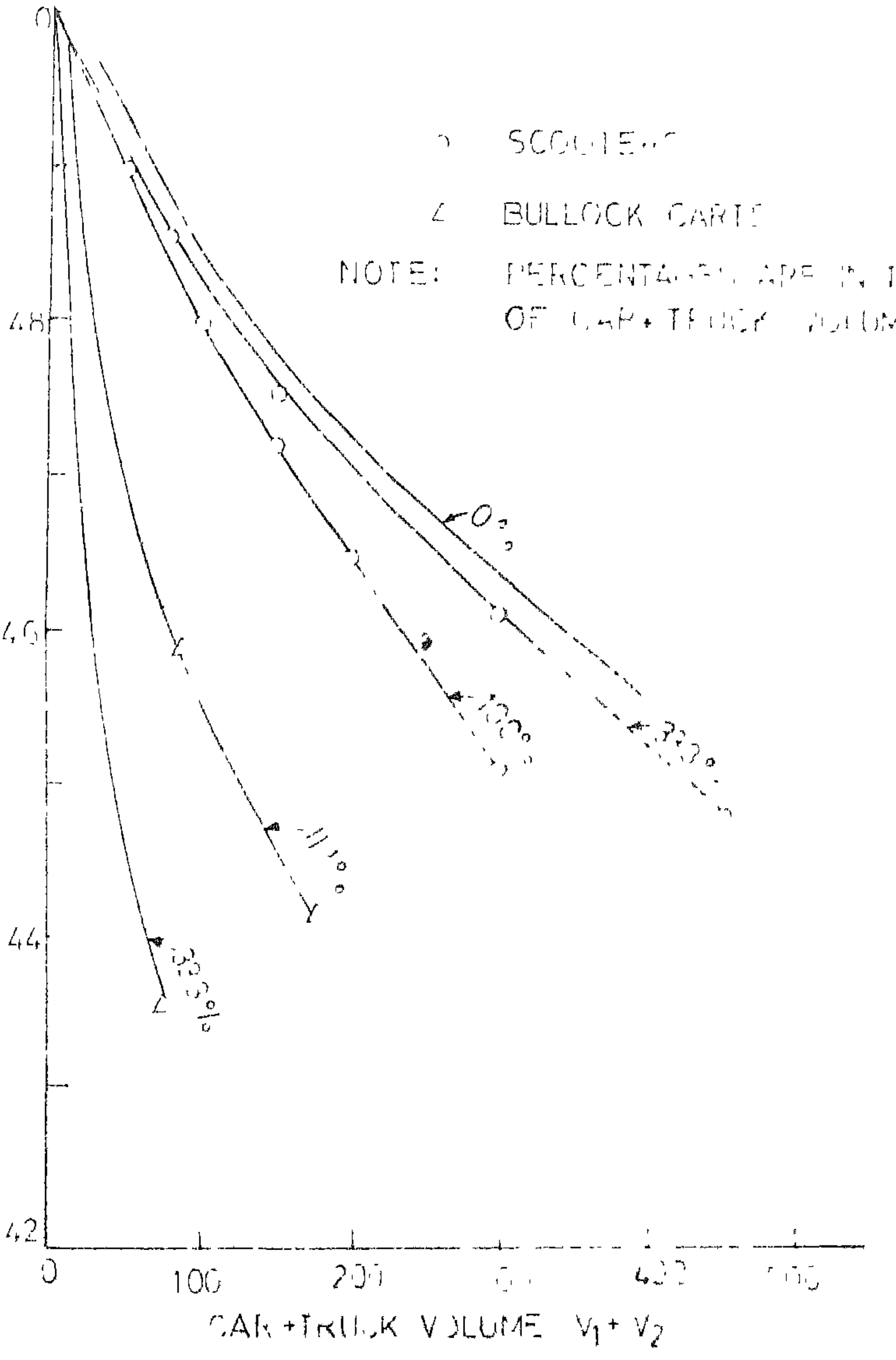
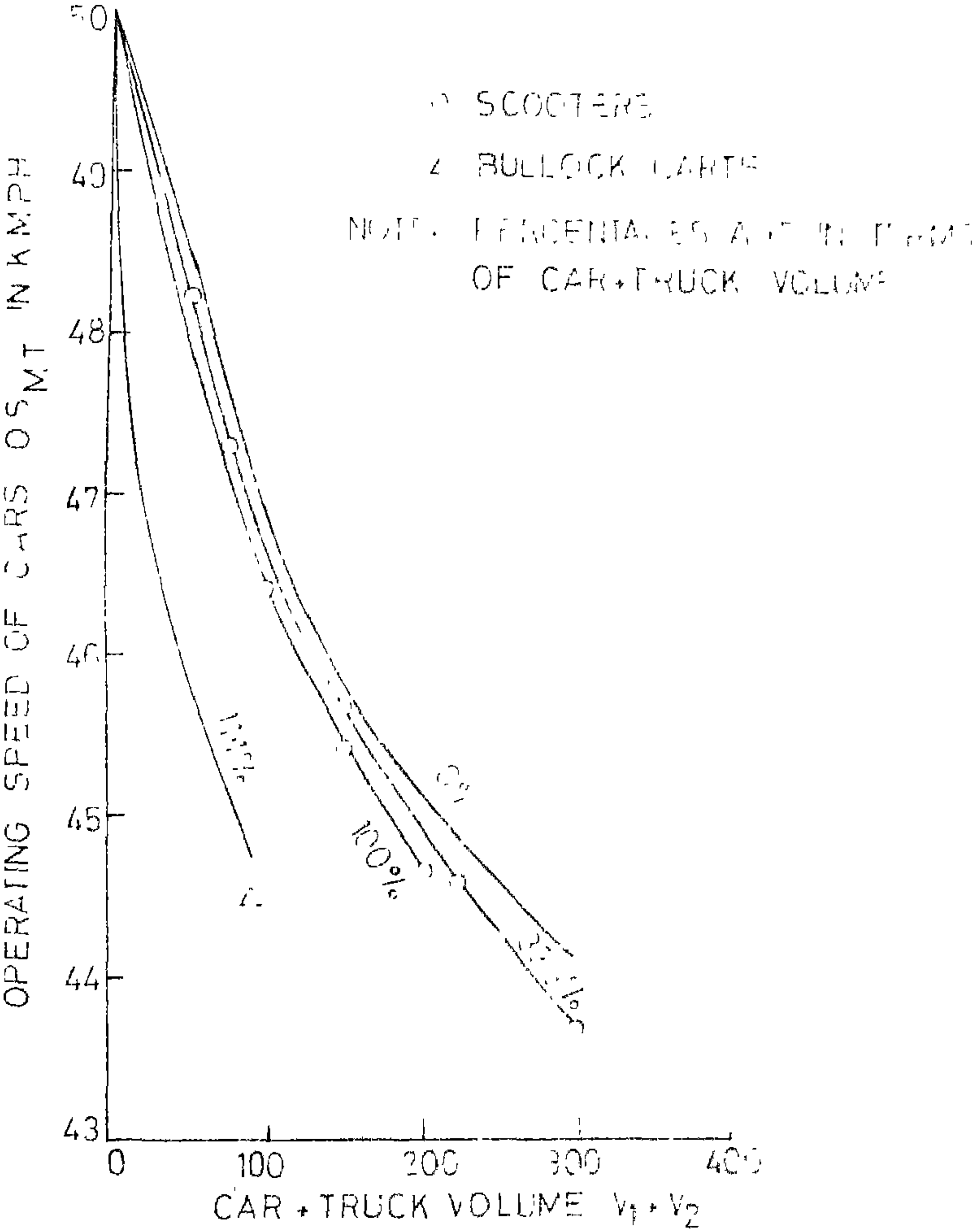


FIG.6.26 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR TRUCK RATIO 50/50



G·6·27 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR-TRUCK RATIO 70/30



IG.6·28 SPEED VOLUME RELATIONSHIP FOR THREE VEHICLE COMBINATIONS HAVING CAR TRUCK RATIO 50/50

the conditional multiplicative model (Eq. 6.36).

6.6 Interaction Between Six Different Categories of Vehicles

On the basis of earlier results, a generalised conditional multiplicative model was formulated to define the operating speed of cars in mixed traffic having several categories of vehicles (NCAT), namely ;

$$OS_{MTNCAT} = OS_{HT} (V_1) . IF_2 (V_1 , P_2) \prod_{i=3}^{NCAT} IF_i[(V_1+V_2), P'_i, CAT_i] \qquad (6.37)$$

where P'_i is the percentage of vehicles of i th category in terms of car truck volume .

It may be noted that homogeneous traffic, two or three vehicle combinations considered earlier are special cases of this model.

Simulation analysis was carried out for six vehicle combinations only for two specific compositions and for different volume levels. The results were compared with the operating speeds predicted by the conditional multiplicative model (Table 6.7). The absolute errors had an average value of 0.3 percent with a maximum of around 0.6 percent. Even though they are slightly larger than for three vehicle combinations (Table 6.6) , these results are considered to be quite satisfactory.

TABLE 6.7 COMPARISON OF SIMULATION RESULTS WITH THOSE COMPUTED FROM MULTIPLICATIVE MODEL (EQ.6.37) FOR COMBINATION OF SIX VEHICLES.

Composition in Percent		Interaction Factors at Various Volume Levels			
			Volume Levels		
			100	200	300
Cars	=17.5				
Trucks	= 7.5	IF_2	0.9925	0.9870	0.9684
Tongas	= 6.0	IF_3	0.9906	0.9821	0.9743
Bullock Carts	= 4.0	IF_4	0.9469	0.9393	0.9323
Scooters	=15.0	IF_5	0.9970	0.9966	0.9962
Bicycles	=50.0	IF_6	0.9916	0.9897	0.9875
OS_{HT}			49.97	49.93	49.87
OS_{MT3}	from Model		46.34	44.84	43.80
OS_{MT3}	from Simulation		46.17	45.10	44.07
% Error			0.37	-0.58	-0.61
Cars	=15.0				
Trucks	=15.0	IF_2	0.9868	0.9646	0.9469
Tongas	= 6.0	IF_3	0.9895	0.9802	0.9719
Bullock Carts	= 4.0	IF_4	0.9531	0.9449	0.9376
Scooters	=10.0	IF_5	0.9983	0.9980	0.9977
Bicycles	=50	IF_6	0.9921	0.9921	0.9879
OS_{HT}			49.97	49.94	49.89
OS_{MT3}	from Model		46.05	44.09	43.63
OS_{MT3}	from Simulation		46.21	44.16	43.61
% Error			-0.35	-0.15	0.05

The results of this section indicate that separability of the interacting factors (IF_i) in terms of a conditional multiplicative model is valid. So the generalised conditional multiplicative model can be used for estimating the operating speed of cars in mixed traffic flow for different combinations, compositions and volume levels, and perhaps even for different free speed distributions of vehicles.

6.7 Characterisation of Mixed Traffic Flow

6.7.1 Introduction

The characterisation of mixed traffic flow is quite complex due to wide variations of characteristics of vehicles in countries like India. The characterisation of mixed vehicular traffic is normally done in terms of passenger car equivalents (PCES) of different vehicles. The PCES are defined as, "Number of passenger cars displaced in the traffic flow by another vehicle under the prevailing roadway and traffic conditions". In this study, the mixed flow is characterised in terms of PCES which express the relative effects of each type of vehicle on the operating speed of passenger cars. The speed volume relationships derived in Secs. 6.4 to 6.6 were used for characterisation of traffic flow.

The operating speed of cars in the mixed flow is a nonlinear function of traffic volume and its composition. The equivalence between mixed traffic and homogeneous traffic is in terms of the level of service. Hence PCUS equivalent to any given mixed traffic is represented by the volume of homogeneous car traffic which has the same operating speed. Knowing the composition of mixed traffic, the PCES for different categories of vehicles can be derived from the PCUS for mixed traffic.

6.7.2 Characterisation of Two Vehicle Combination

PCUS are defined explicitly in terms of OS_{MT2} by Eqs.6.5 and 6.6, viz.,

$$OS_{MT} = 50 - 0.0014(PCUS) - 0.000019\,(PCUS)^2 \qquad \text{for } PCUS \leq 150 \qquad (6.38)$$

and

$$OS_{MT} = 50.875 - 0.0101\,(PCUS) \qquad \text{for } 150 \leq PCUS \leq 650 \qquad (6.39)$$

Given OS_{MT2}, the corresponding PCUS(2) may be established from these equations, say,

$$PCUS_{(2)} = f^{-1}[(OS_{MT2})] = f^{-1}\left[OS_{HT}(V_1).IF(V_1, P_2, CAT)\right] \qquad (6.40)$$

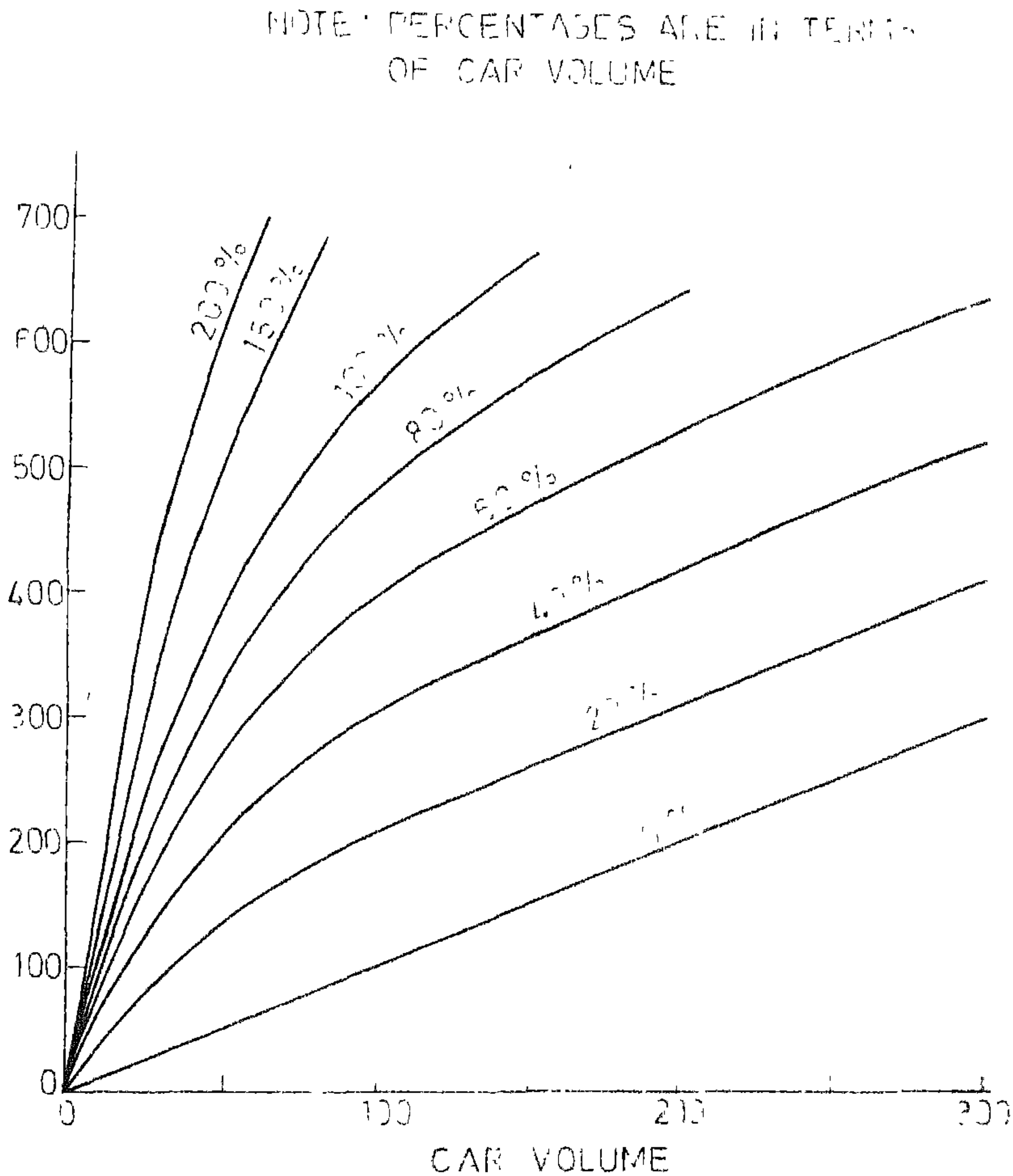
where $PCUS_{(2)}$ is the equivalent car volume for a two vehicle combination. The PCUS of various car truck combinations at different volume levels and compositions are shown in Fig. 6.29.

The PCES of the second category of vehicles in a mix can be estimated as follows:

$$PCES_{(2)} = \frac{PCUS_{(2)} - V_1}{(P_2 V_1/100)} \qquad (6.41)$$

where P_2 is the percentage of second category of vehicle in terms of car volume V_1. The PCES for different categories of vehicles in 2 vehicle combinations are shown in Figs. 6.30 to 6.34 at different volume levels and proportions. It is assumed that jamming occurs when PCUS equal 700 VPH.

The results indicate that for trucks, bullock carts and tongas, the PCES reduce with increased car volume. Furthermore, they decrease as their proportion in the mix increases. This is because the relative interaction effect of these vehicles are smaller at higher volumes (Fig.6.12) of cars and themselves. On the other hand for scooters and bicycles, the PCES increase with car volume and with higher proportion of scooters and bicycles in the mix. There seems to be a tendency for the PCES to approach 1 at higher volume of cars and higher proportion of second category of vehicles, viz., for wide base vehicles like trucks, bullock



.6·29 EQUIVALENT CAR VOLUME (PCUS) OF DIFFERENT CAR TRUCK COMBINATIONS

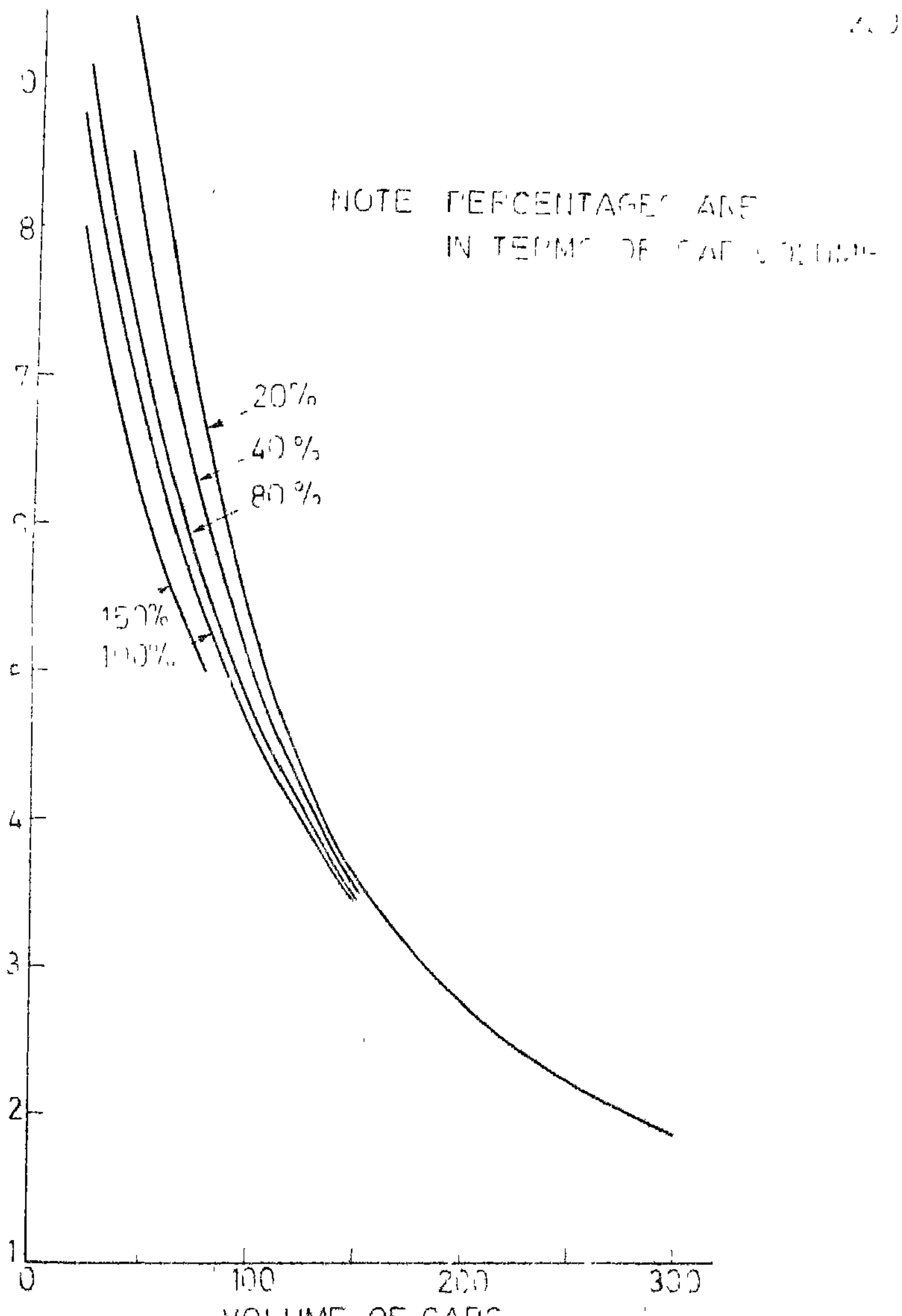


FIG. 6.30 PCES OF TRUCKS IN CAR TRUCK COMBINATIONS

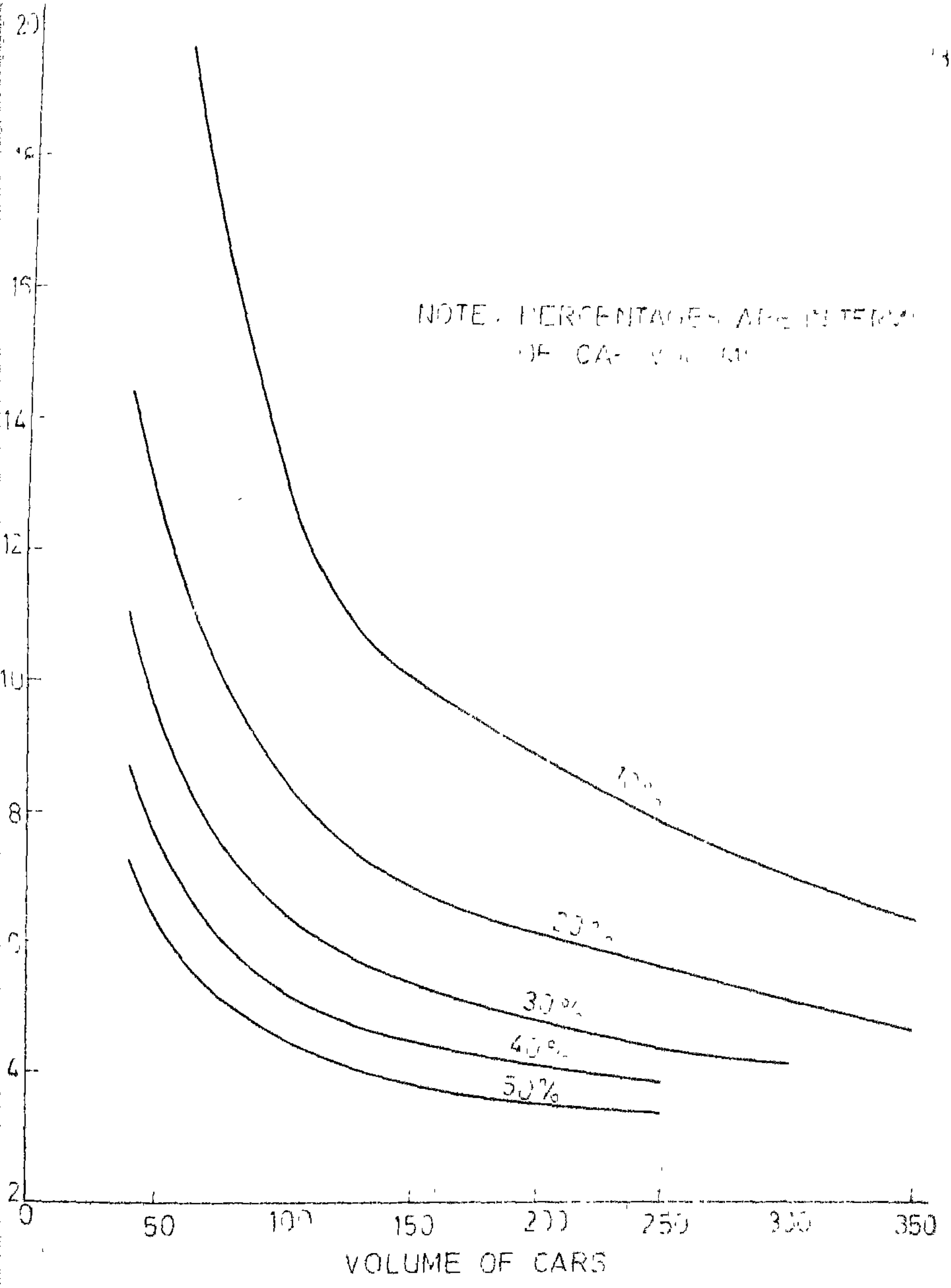


FIG. 6.31 P C E S OF TONGAS IN CAR TONGA COMBINATIONS

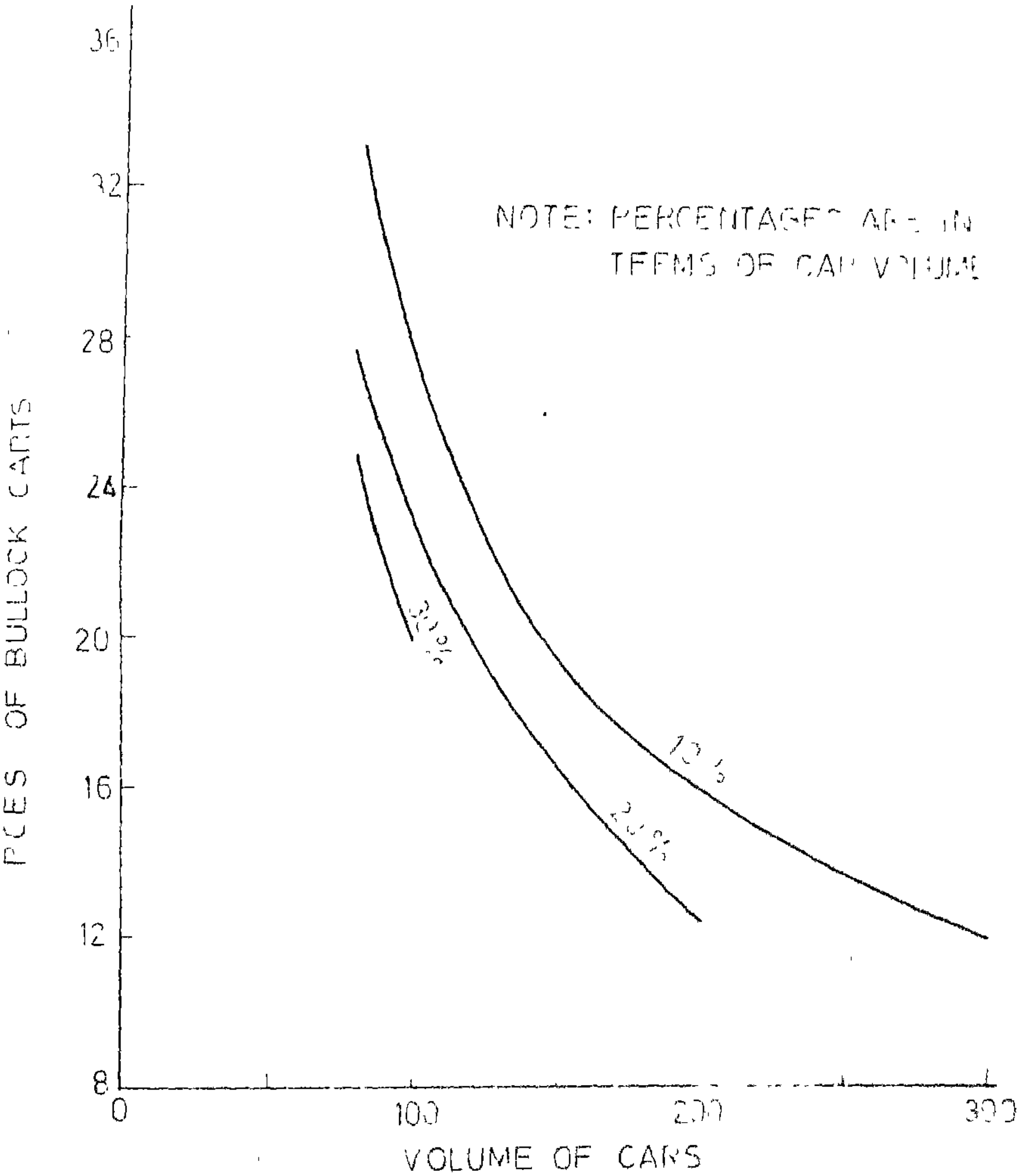


FIG 6·32 PCES OF BULLOCK CARTS IN CAR BULLOCK CART COMBINATIONS

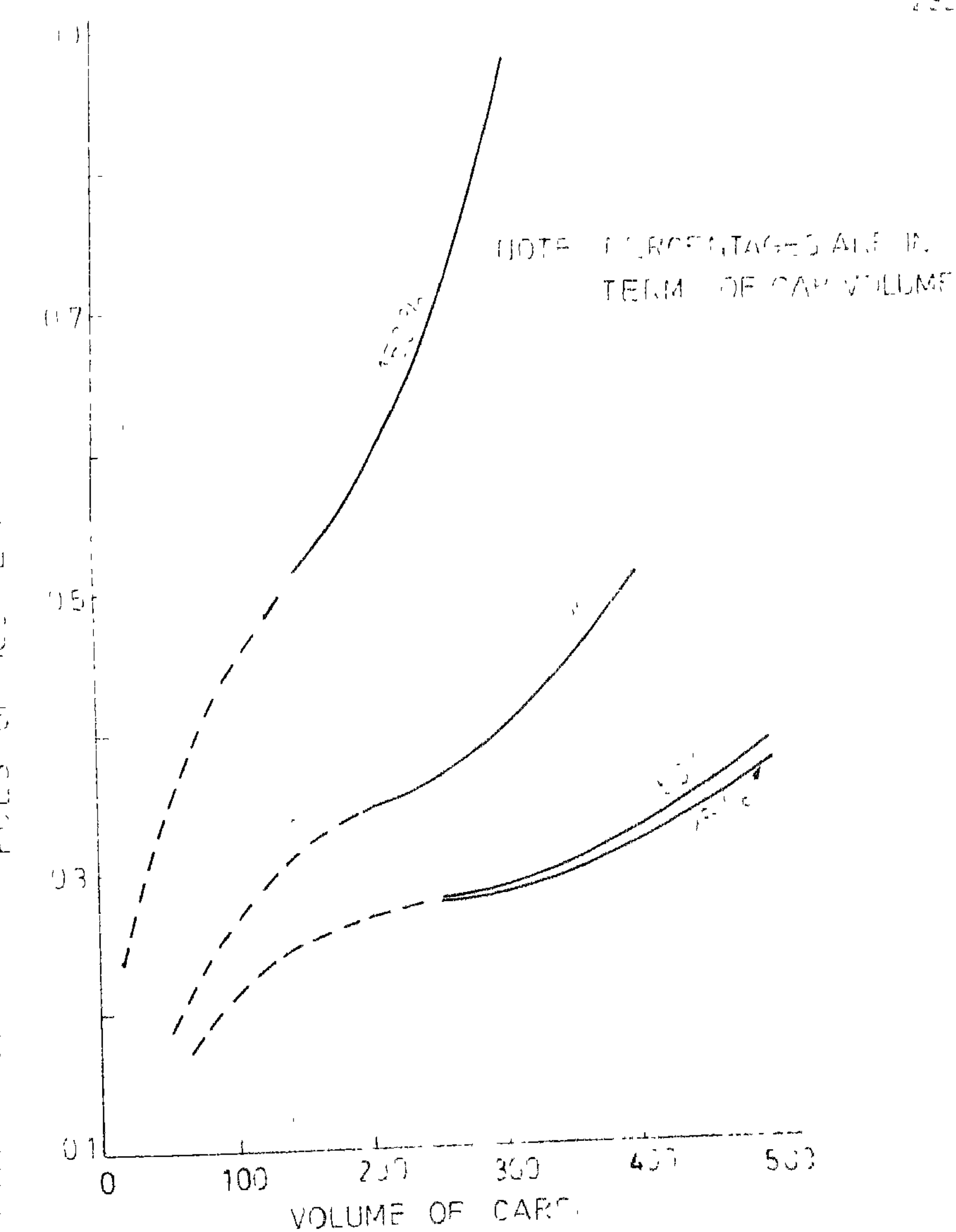
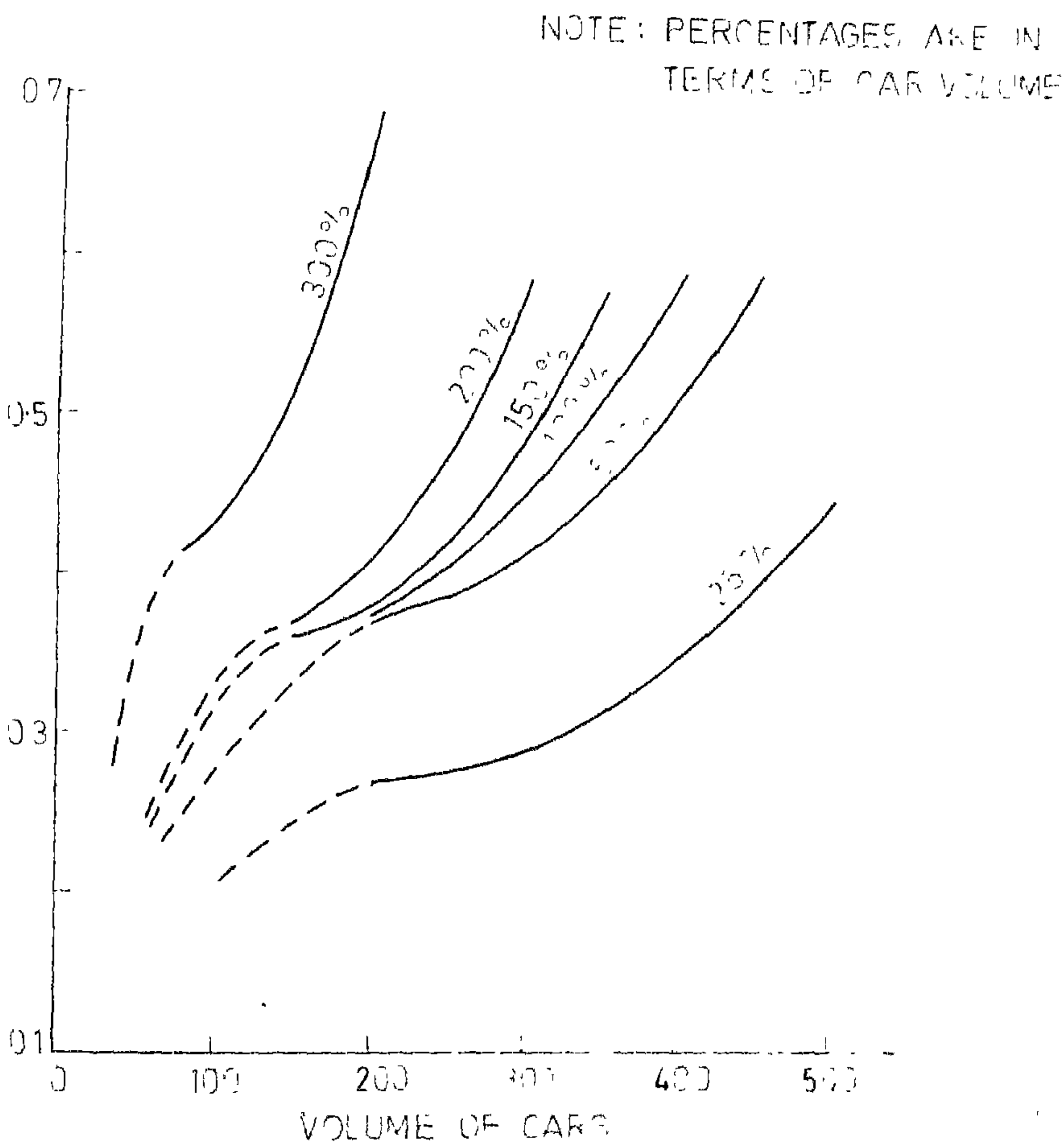


FIG. 6.33 PCES OF SCOOTERS IN CAR SCOOTER COMBINATIONS



IG.6·34 PCES OF BICYCLES IN CAR BICYCLE COMBINATIONS

carts and tongas, which have a PCES larger than 1, the PCES decreases and in case of two wheelers like scooters and bicycles, which have a PCES of less than 1, the PCES increases with car volume and proportion of the second vehicle. The effect of the proportion of vehicles is much smaller in case of PCES of trucks than in case of other vehicles.

The range of PCES for different categories of vehicles are given in Table 6.8. The constant values suggested by CRRI and MOT are valid atmost over a small range of combinations and these ranges are also indicated in Table 6.8. It is clear from Figs. 6.30 to 6.34 and Table 6.8 that PCES are variable functions of composition and volume level of mixed traffic.

6.7.3 Characterisation of Three Vehicle Combination

When there are three categories of vehicles in the mix, the operating speed of cars for car truck combinations (OS_{MT2}) and three vehicle combinations (OS_{MT3}) can be derived respectively by Eq. 6.28 and 6.36. The PCUS for mixed flow for car truck combinations $PCUS_{(2)}$ and for three vehicle combinations $PCUS_{(3)}$ can be

TABLE 6.8 RANGE OF PCES FOR DIFFERENT CATEGORIES OF VEHICLES AT VARYING TRAFFIC COMPOSITIONS

Category of Vehicle	Passenger Car Equivalents (PCES)				
	CRRI	MOT	From Simulation Analysis		
			Proportion as Percentage of Car Volume	Car Volume Range in VPH	PCES Range
Trucks	2.7	3.0	20 - 100	50 - 150	9.0 - 3.5
			150	25 - 75	8.0 - 5.0
Tongas	2.6	6.0	10 - 20	50 - 350	20.0 - 4.7
			20 - 30	50 - 300	13.0 - 4.1
			30 - 50	50 - 250	3.7 - 3.5
Bullock Carts	10.7	6.0	10 - 20	75 - 300	33.0 -12.0
Scooters	0.2	1.0	25 - 50	50 - 500	0.2 - 0.4
			50 - 100	50 - 400	0.2 - 0.5
			100 - 150	50 - 300	0.2 - 0.37
Bicycles	0.4	1.0	25	50 - 500	0.2 - 0.45
			25 - 50	50 - 450	0.2 - 0.60
			50 - 100	50 - 400	0.2 - 0.60
			100 - 150	50 - 350	0.2 - 0.60
			150 - 200	50 - 300	0.2 - 0.60
			200 - 300	50 - 200	0.2 - 0.70

determined from the respective operating speeds of cars by the equations

$$PCUS_{(2)} = f^{-1} (OS_{MT2}) \qquad (6.42)$$

and $$PCUS_{(3)} = f^{-1} (OS_{MT3}) \qquad (6.43)$$

The PCES for trucks can be estimated from Eq.6.41. The PCES of the third vehicle can be determined as follows:

$$PCES_{(3)} = \frac{PCUS_{(3)} - PCUS_{(2)}}{V_3} \qquad (6.44)$$

where volume of third vehicle

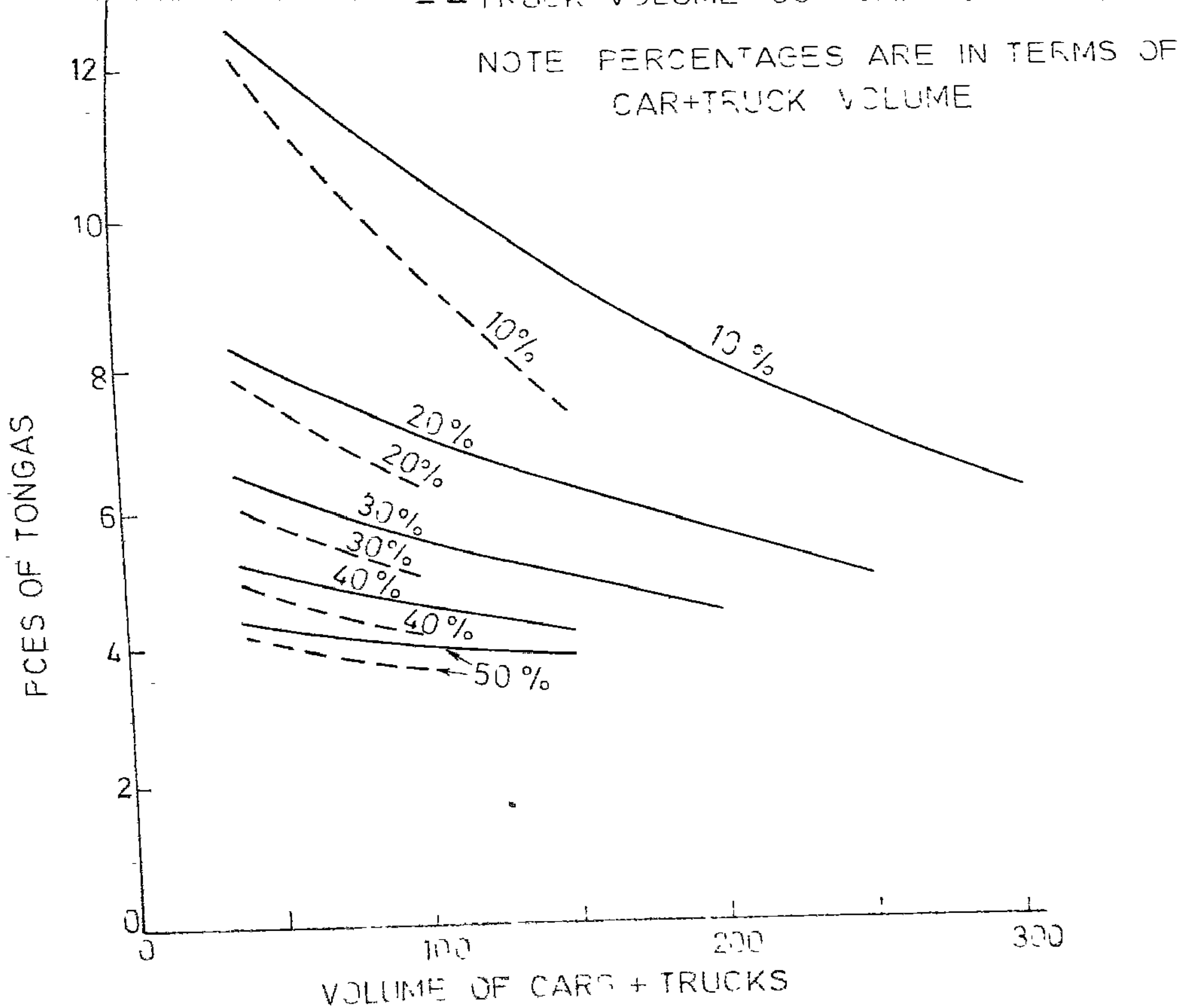
$$V_3 = P_3' (V_1 + \frac{P_2 V_1}{100}) \qquad (6.45)$$

The PCES of the third category of vehicles for different percentages of car truck volume are indicated in Figs. 6.35 to 6.38 and these may be used as design charts when appropriate. The variations of $PCES_{(3)}$ are similar to those of the same category for two vehicle combinations.

6.7.4 Characterisation of Multivehicle Combinations

When the number of categories of vehicles in the mix exceed three, they can be considered in a specified order and PCES for each category of vehiclescan be calculated as follows:

$$PCES_{(i)} = \frac{PCUS_{(i)} - PCUS_{(i-1)}}{V_i} \qquad (6.46)$$



FIG·6·35 PCES OF TONGAS IN CAR TRUCK AND TONGA COMBINATIONS

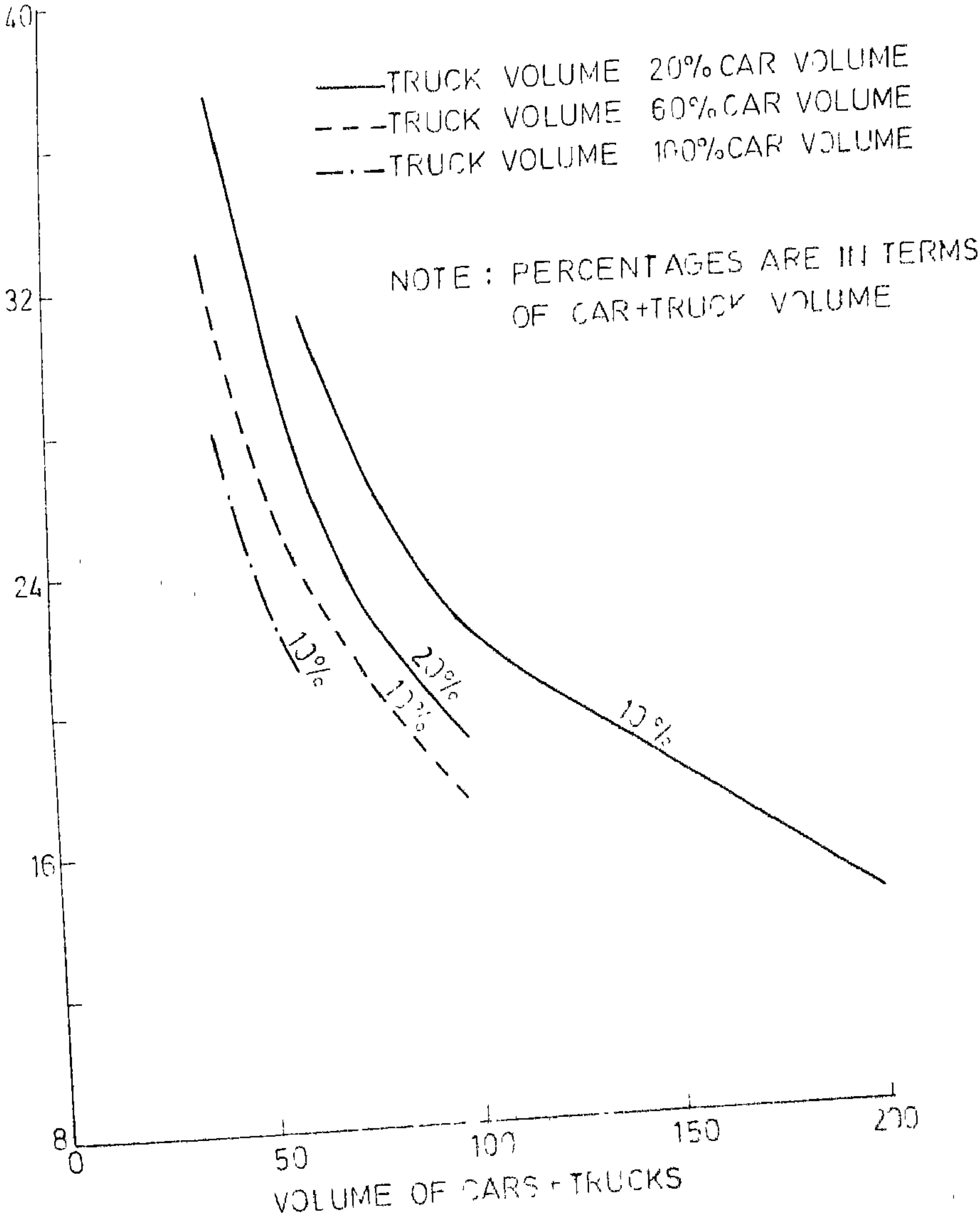


FIG.6.36 PCES OF BULLOCK CARTS IN CAR TRUCK AND BULLOCK CART COMBINATIONS

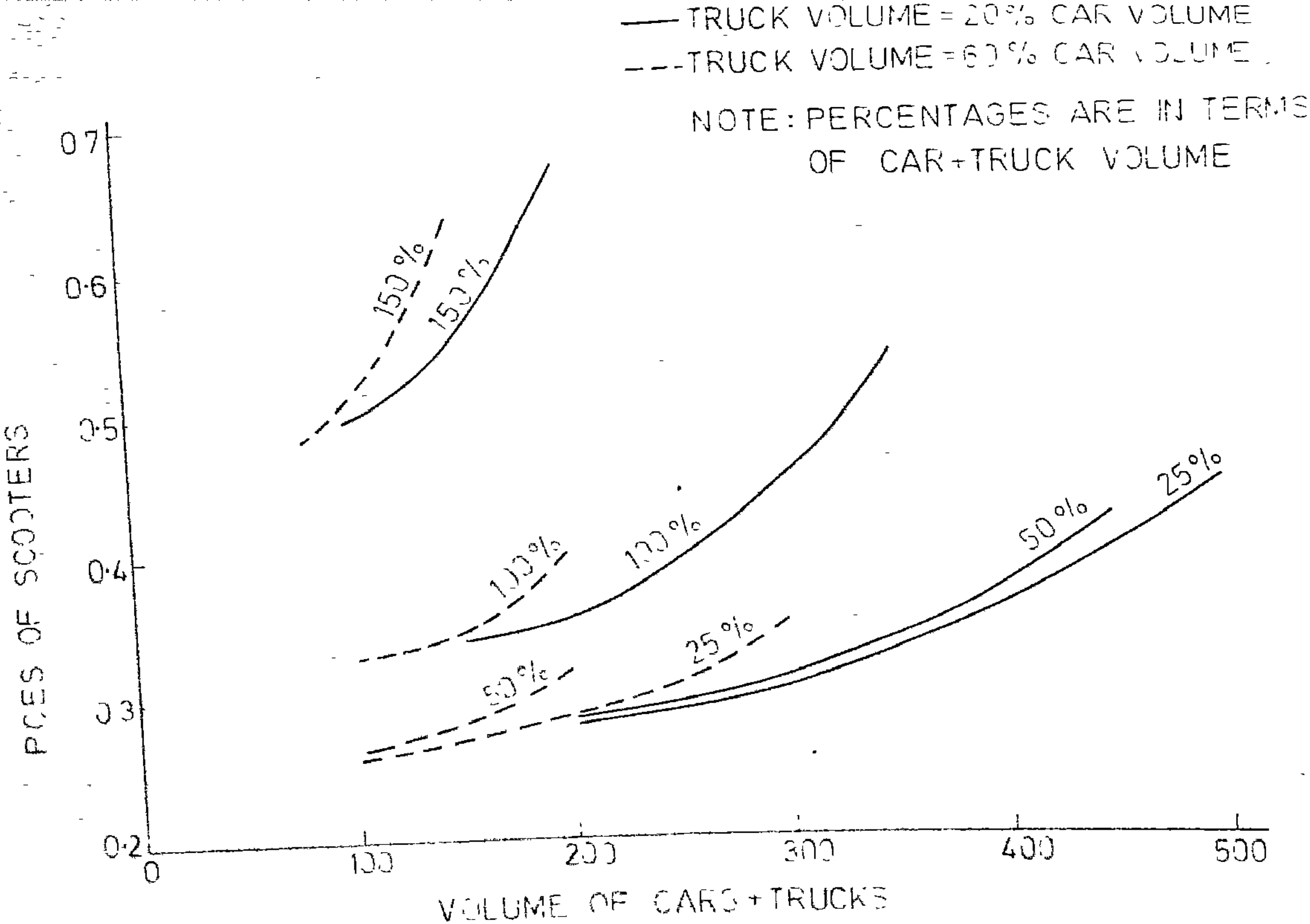


FIG 6.37 PCES OF SCOOTERS IN CAR TRUCK AND SCOOTER COMBINATIONS

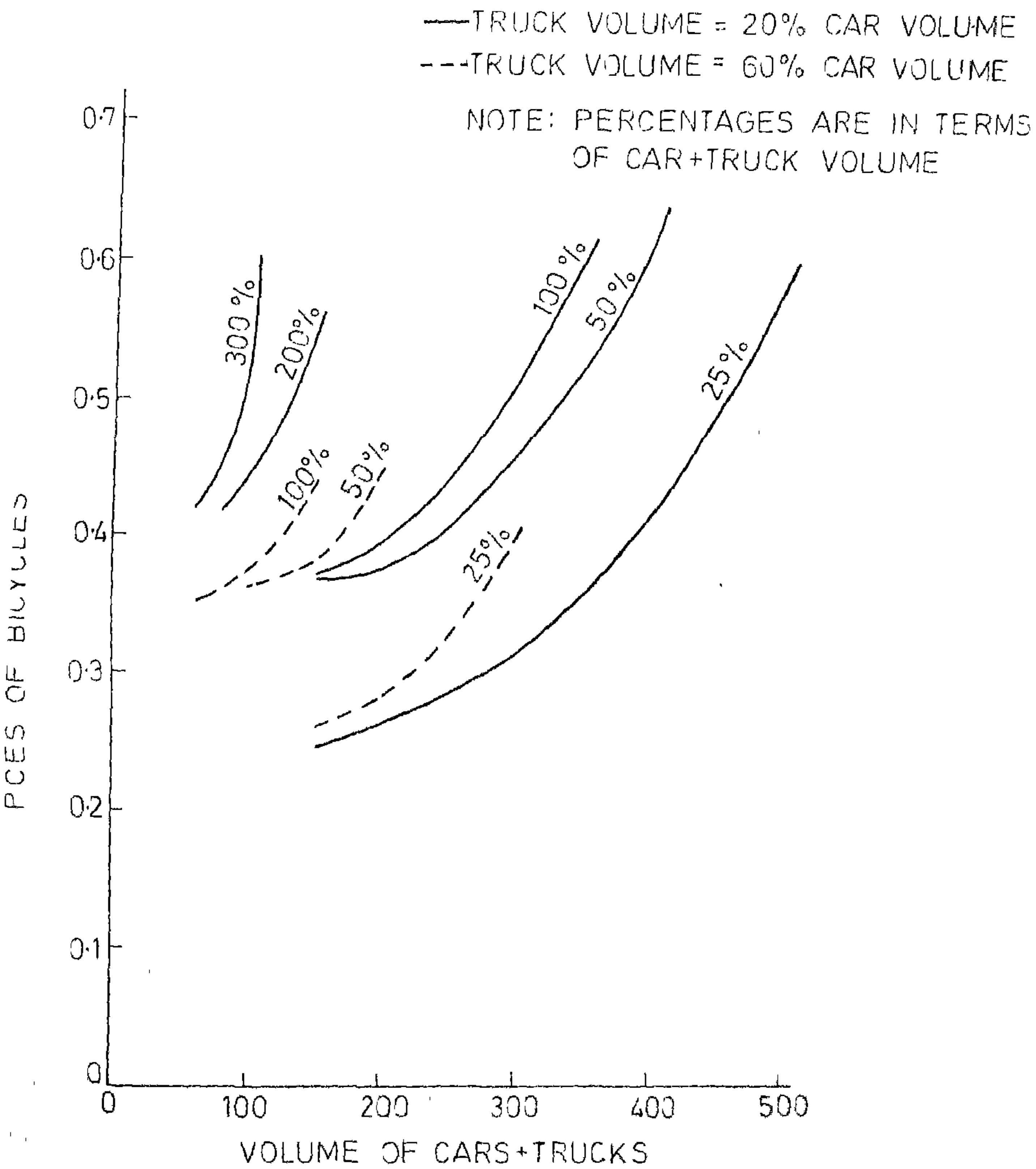


FIG.6.38 PCES OF BICYCLES IN CAR TRUCK AND BICYCLE COMBINATIONS

where volume of i th category of vehicle $V_i = P_i'(V_1 + \frac{V_1 P_2}{100})$.

It should be noted that PCES estimated depend upon the sequential order in which vehicles are considered. It is suggested that car truck combination be considered first; the vehicles of wide base considered next and finally two wheelers. Furthermore, in each subgroup, the vehicles with higher average speeds be considered ahead of vehicles with lower speed, viz., in the order trucks, tongas, bullock carts, scooters and bicycles.

The results of this study indicate that the nonlinear interaction between different categories of vehicles may be represented by a generalised conditional multiplicative model for operating speed of cars which defines the level of service. This can be used to define the equivalent volume of cars (PCU S) for the same level of service and in turn to determine the variable PCES of different categories of vehicles in mixed traffic flow.

7. SUMMARY, CONCLUSIONS AND SUGGESTIONS FOR FUTURE STUDY

7.1 Summary

Characterisation and estimation of traffic are two important aspects in the planning and design of a highway transportation system. In developing countries like India, the traffic is of mixed nature consisting of both slow and fast moving vehicles between which there are wide variations in speed. Generally constant values of PCES of different categories of vehicles are specified by appropriate authorities.

This study considers the nonlinear interaction between the different vehicles in mixed traffic flow situations and attempts to characterise the variations of PCES, in terms of traffic composition and volume. Traffic flow is stochastic in nature, the free speeds are probabilistic and the process is affected by logical decisions concerning acceleration, retardation, overtaking etc. Analytic techniques of traffic flow theory, are suitable for the analysis of simple homogeneous vehicular traffic and it does not seem feasible to use exact analytic procedures for the analysis of complex mixed traffic flow. Computer simulation is adopted in this study to analyse the mixed

traffic flow and infer the PCES under different volumes and compositions of traffic.

Traffic flow on a highway is a stochastic process with seasonal, weekly and daily variations. Using historical monthly and daily traffic data of seven years on five main highways approaching Kanpur, stochastic models have been developed for the monthly and daily traffic data for three categories of goods carriers. Trends, annual, six monthly and weekly cycles may be present in the data series and the residual series can be represented by a first order AR model. Alternatively, monthly series can be represented by a multiplicative seasonal ARIMA model of order $(1, 0, 0) \times (0, 1, 1)_{12}$. Stochastic models can also be used for making forecasts about traffic volumes.

Detailed data were needed for simulation and so field studies were conducted for ten days round the clock on a 2.25 km stretch of Grand Trunk Road. From the observations, peak hourly volumes; variations of volumes within the day of six vehicular types; and interarrival time gap distributions were estimated. The free speeds, delay time and operating speeds of different categories of vehicles were also estimated at varying volume levels and traffic compositions.

A computer simulation model of mixed traffic flow on a two lane highway was formulated. Vehicles move from either direction of the roadway section and scanning is done at one second intervals. Any vehicle in the roadway moves at its free speed where it had enough headway to travel unimpeded. In case it cannot move unimpeded and it has a higher speed than the vehicle ahead of it, then it may try to overtake. Overtaking is possible only if there is no conflict with the opposing traffic stream during overtaking operation. Otherwise the vehicle is forced to reduce its speed to that of the vehicle ahead until sometime later overtaking is possible. The above logic is applied to each of the vehicles in the section and those entering the section for each time interval. As a vehicle leaves the section, time of leaving is noted and its characteristics like travel time, delay time and overall running speed etc., are determined.

Initially historical data of arrival times, traffic composition and system parameters were used to identify the system and validate the system model. The roadway is divided into one metre long sections and vehicles are moved from either direction at one second intervals. Vehicles move at their free speed when there is enough headway to travel unimpeded for two seconds. The change

of speed of vehicles is assumed to be instantaneous and a looking for gap time of two seconds is provided for overtaking operations. The overtaking vehicle accelerates only if the speed difference with the overtaken vehicle is less than 16 kmph. Otherwise overtaking takes place at normal speed. For vehicles being overtaken, minimum spacing related to speed and length of vehicle, is specified. The results from the simulated model were consistent with the observed characteristics and the model is thus validated. Simulation is hence found to be a versatile tool for understanding of the flow process and identifying the system model.

Traffic on a highway is highly nonstationary with hourly trends, and cycles, and persistence. However, for design considerations a stationary peak rate corresponding to a specified level of risk may be assumed. So this study deals with simulation of stationary traffic flow. It may be noted that when appropriate, more complicated nonstationary processes can be simulated to derive comparable results. Generated data were used in simulation of stationary mixed traffic flow at different volumes and compositions. When generated data are used in computer simulation, it is necessary to initialise the system. In this study initially the stretch of the road was assumed to

be empty. The process was run until a steady state was reached. This initial time period varied generally with the volume level, being more for lower traffic volumes. Simulation was carried out further for a sufficiently long time for estimation of the characteristics of the process.

Simulation was performed for homogeneous, two, three and six vehicle combinations for specific compositions and volume levels. The characteristics observed include (i) proportion of delayed vehicles; (ii) average delay time; (iii) operating speed of different categories of vehicles; and (iv) density of section.

The study indicates that, a nonlinear interaction between different categories of vehicles in mixed traffic flow can be analysed by simulation; the capacity and operating characteristics of highways can be established; mathematical models can be formulated for representing the interactions; and PCUS for mixed traffic and PCES for each category can be derived as a function of traffic volume and composition.

7.2 Conclusions

On the basis of this study, the following conclusions can be drawn:

(i) Traffic flow on a highway is a stochastic process with seasonal, weekly and daily variations. Trends and annual, six monthly and weekly cycles may be present in the data series and the residual series can be represented by a first order AR model. Alternatively, the monthly series can be represented by a multiplicative seasonal ARIMA model of order $(1, 0, 0) \times (0, 1, 1)_{12}$. Stochastic models can be used for making forecasts about traffic volumes.

(ii) A simulation model which includes unimpeded, overtaking and restrained stream logics is formulated using field data. Parameters of the model are estimated and the simulation model is validated. Simulation leads to a better understanding of the components and interactions of the complex process. It facilitates mathematical modelling of the system and the process.

(iii) Traffic flow can be classified into four distinct levels of service, viz., free flow, stable flow, unstable flow and forced flow. Simulation helps in identification of these levels for homogeneous and mixed traffic and also for estimation of highway capacity.

(iv) The level of service can be specified in terms of operating speed of passenger cars in mixed traffic flow. Surfaces of equal levels of service can also be derived for mixed traffic flow.

(v) From results of simulation, it is possible to derive regression equations relating volume, operating speed and/or density. Relationships for homogeneous and two vehicle combinations are given in this study.

(vi) Simulation analysis of homogeneous car traffic and of two vehicle combinations indicate that interaction is a function of volume level and traffic composition. The operating speed of cars in mixed flow (OS_{MT2}) is a product of operating speed of cars alone (OS_{HT}) and the interaction factor, IF, which depends upon the volume of cars (V_1) , and the type (CAT) and proportion of vehicles of second category. Mathematical relationships for IF's have also been derived.

(vii) For three or more vehicle combinations a generalised conditional multiplicative model with separable interacting factors, IF's, of the following type has been formulated.

$$OS_{MTN\,CAT} = OS_{HT}\ (V_1).\ IF_2(V_1,P_2)\ .\prod_{i=3}^{NCAT} IF_i\,(V_1+V_2,P_i',CAT_i)$$

where $OS_{MTN\,CAT}$ = operating speed of cars in mixed flow

having NCAT categories of vehicles; P_2 = truck volume V_2 expressed as a percentage of car volume V_1 ; P_i' = proportion of ith category of vehicle (CAT_i) expressed as percentage of car truck volume ($V_1 + V_2$) ; and IF_i = interaction factor as derived for two vehicle combinations but in terms of ($V_1 + V_2$) and P_i' .

The above model has been validated from simulation results for three and six vehicle combinations for a number of compositions and volume levels.

(viii) The generalised model can be used for estimating the operating speed of cars in mixed traffic flow and also for determining equivalent passenger car volume (PCU S). This can in turn be used to derive PCES for different categories of vehicles. Design charts for PCES for two and three vehicle combinations have also been drawn.

(ix) For trucks, tongas and bullock carts, the PCES decrease with increased car volume and with increased proportion of these vehicles in the mix. On the other hand for scooters and bicycles, PCES increase with car volume and with higher proportion of scooters and bicycles in the mix. There seems to be a tendency for the PCES to approach unity at higher volume of cars and higher proportion of second category of vehicles.

(x) The study hence clearly demonstrates the nonlinear interaction in mixed traffic flow; that these interactions can be defined in terms of the generalised model; and that PCU S and PCES can be derived from such a model.

(xi) It seems possible to combine stochastic modelling and simulation results for the design of highways. For example, from stochastic models of different categories of mixed traffic, the ADT of each category for a given level of risk may be determined. From the composition of vehicles at critical period(s) and the derived ADT's, the volume of each category of vehicle in the critical period(s) may be estimated. From the PCES for different categories of vehicles for given volume and composition, the PCUS of mixed traffic at critical period(s) can be determined. The highways can then be appropriately designed.

7.3 Suggestions for Future Study

Based on the results of this study, the following suggestions are made for future work in this area:

(i) This study deals with the simulation of mixed traffic flow on a two lane highway and is limited to specific compositions, free speed distributions, straight section with no lateral obstructions and equal directional

distribution of traffic. While the results of the study indicate the nonlinear interactions, it seems necessary to extend the study to different compositions, different free speed distributions of vehicles, varying geometrics like grades, curves, lateral obstructions etc., varying directional distribution of traffic, breakdown of vehicles etc., and perhaps for intersections and intercity traffic. The results of the study including the mathematical models may need modifications because of these factors.

(ii) It seems necessary to develop a problem oriented Simulation Language for the simulation of mixed traffic flow in multilane highways. Scuh a programme will be helpful in simulating complex mixed vehicular traffic on highways with different number of lanes and other geometrics.

(iii) Stochastic modelling of some categories of traffic has been indicated in this study. It seems desirable to develop multivariate stochastic models which relate temporal variations of mixed traffic and preserve corre-lations among the different categories of vehicles. Using such models, data can be generated and critical periods may be identified. Using detailed generated data for these critical periods, the mixed traffic flow can be simulated for a particular design of highway system. The

risk associated with the design may be evaluated and this can be used in an iterative design of the system.

(iv) The capacity of highways is generally given in terms of homogeneous traffic. Using simulation, it is possible to derive the capacity and level surfaces for different combinations of mixed traffic. For combinations of more than three categories of vehicles, a graphical representation is not possible and it may be necessary to either prepare sets of graphs or combine the vehicles to atmost three groups of vehicle combinations. The variability of capacity and level surfaces as a function of the free speed distributions, geometrics etc., also need further investigation.

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